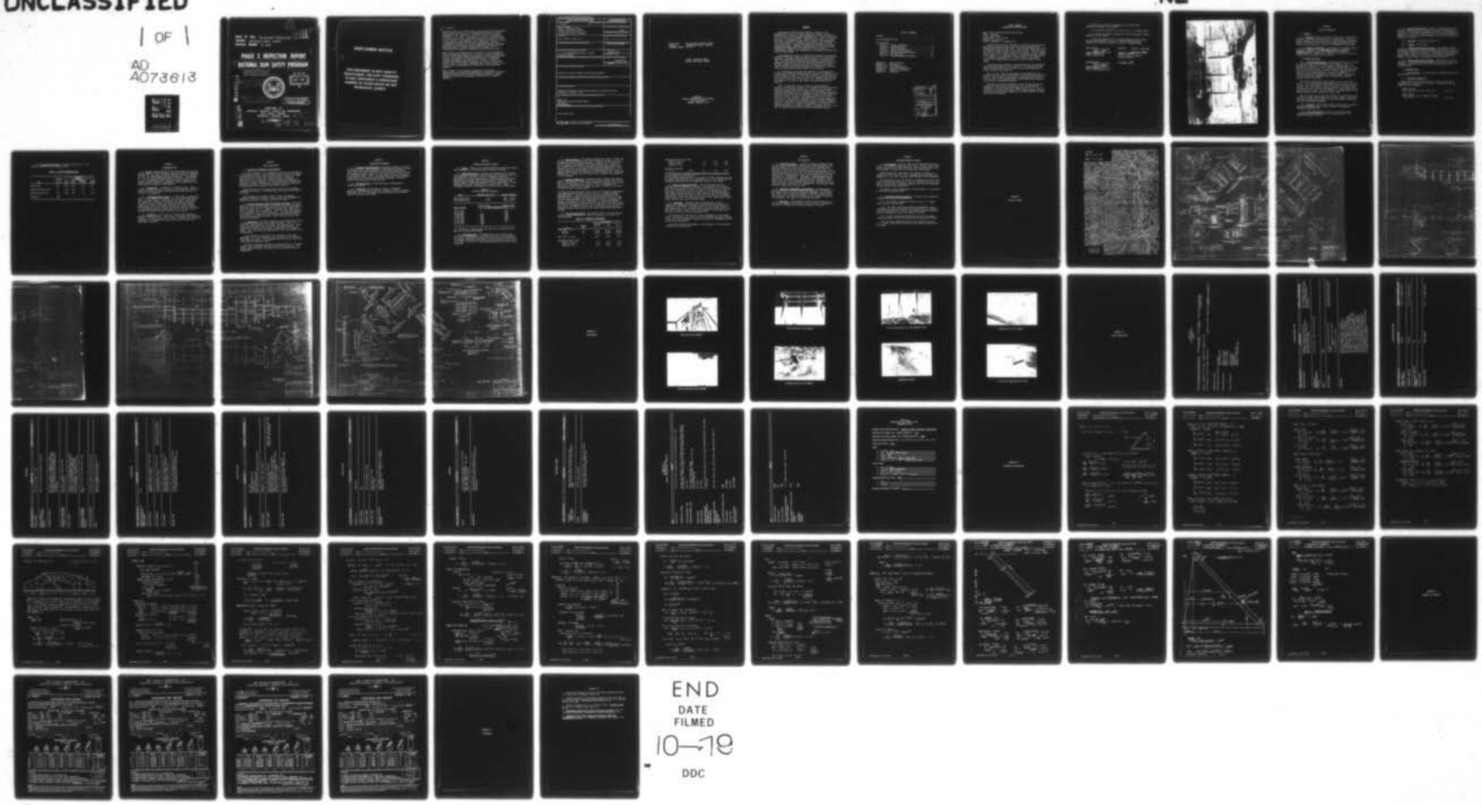
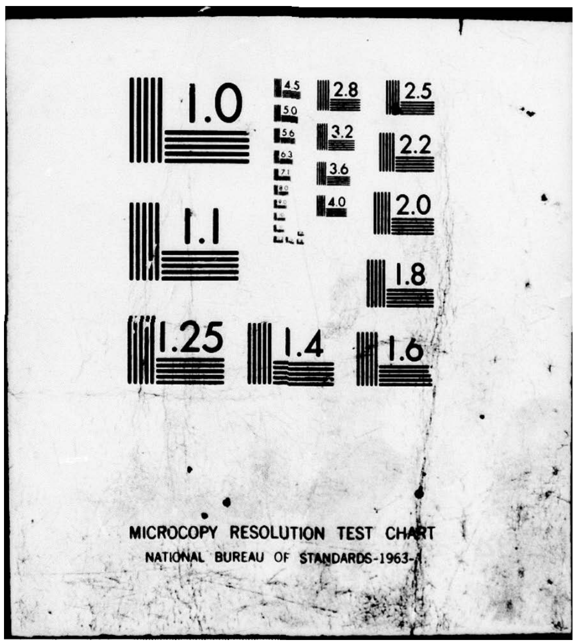
AD-A073 613   ARMY ENGINEERING DISTRICT NORFOLK VA   F/G 13/2
NATIONAL DAM SAFETY PROGRAM. FALLING CREEK FILTRATION PLANT. IN--ETC(U)
JAN 79   J A WALSH

UNCLASSIFIED   NL

1 OF 1
AD
A073613



END
DATE
FILMED
10-79
DDC


1.0
4.5
5.0
5.6
6.3
7.1
2.8
3.2
3.6
4.0
2.5
2.2
2.0
1.1
1.8
1.25
1.4
1.6
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

LEVEL II

Name Of Dam: FALLING CREEK FILTRATION PLANT
Location: CHESTERFIELD COUNTY, VIRGINIA
Inventory Number: VA 04115

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

Falling Creek Filtration Plant.
Inventory Number: VA-04115
Chesterfield County, Virginia.
Phase I Inspection Report.

Final rept.,

James A. Walsh

PREPARED BY
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

72p.

JANUARY 1979

411 030

DISCLAIMER NOTICE

20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER VA 04115	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Phase I Inspection Report National Dam Safety Program Falling Creek Filtration Plant Chesterfield County, Virginia		5. TYPE OF REPORT & PERIOD COVERED Final 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) NAO - James A. Walsh, P.E.		8. CONTRACT OR GRANT NUMBER(s) NAO --------------------
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineering District, Norfolk 803 Front Street Norfolk, VA 23510		12. REPORT DATE January 1979 13. NUMBER OF PAGES
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Copies are obtainable from National Technical Information Service, Springfield, Virginia 22151

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dams - VA
National Dam Safety Program Phase I
Dam Safety
Dam Inspection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

(See reverse side)

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

NAME OF DAM: FALLING CREEK FILTRATION PLANT
LOCATION: CHESTERFIELD COUNTY, VIRGINIA
INVENTORY NUMBER: VA 04115

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of the Chief of Engineer, Washington, D.C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the genral condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations testing and detailed computational evaluations are beyond the scope of a Phase I investigation, however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (flood discharge, that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region) or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the design flood should not be interpreted as necessarily posing a highly inadequate condition. The design flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

TABLE OF CONTENTS

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Falling Creek Filtration Plant
State: Virginia
County: Chesterfield
Coordinates: 3718.1 7950.2
Stream: Falling Creek
Date of Inspection: 1 November 1978

Falling Creek Filtration Plant Dam is a 207-foot long and 30-foot high concrete buttressed dam, sited three miles upstream of the Richmond-Petersburg Turnpike (I-95). This dam was constructed in 1952 and raised to elevation 100.0 (top of dam) in 1956. Plans were found for both the initial construction and the additions. A set of design notes, done in 1956, contain a check of the original dam and a design of the addition. An independent Phase I inspection was performed by J. K. Timmons & Associates, Inc., in March 1978, at the owners request. The Timmons Report is used as a basis for much of this report.

During the visual inspection, and additional seep was discovered which the Timmons Report does not refer to. The concrete spillway, slabs, buttress and joints were found to be in same condition as during the March 1978 inspection.

The 1/2 P.M.F., which is considered the appropriate spillway design flood, will reach elevation 106.5, or 6.5 feet above the top of the dam. The spillway is only capable of passing 21% of this flow. Since the dam is an intermediate-size, significant-hazard structure, the spillway is considered inadequate.

The stability analysis performed in 1956 indicates the dam was designed for a water level of elevation 95.0, or, normal flow conditions. Since the appropriate design flow (1/2 P. M. F.) will exceed this elevation by at least 8 feet, further analysis should be performed by a professional engineer at the owners expense to verify the structural stability.

In addition to the above recommendation, the owner should immediately implement the following:

1. The remedial treatment as outlined in the J. K. Timmons Report.

2. A plan of preventive maintenance for the sluice gates, lifting mechanism, and the valves.

3. The seeps identified in the visual inspection of this report and and those identified in the J. K. Timmons Report should be monitored and treated as specified in the J. K. Timmons report.

Submitted By:
Original signed by
JAMES A. WALSH

JAMES A. WALSH, P.E.
Chief, Design Branch

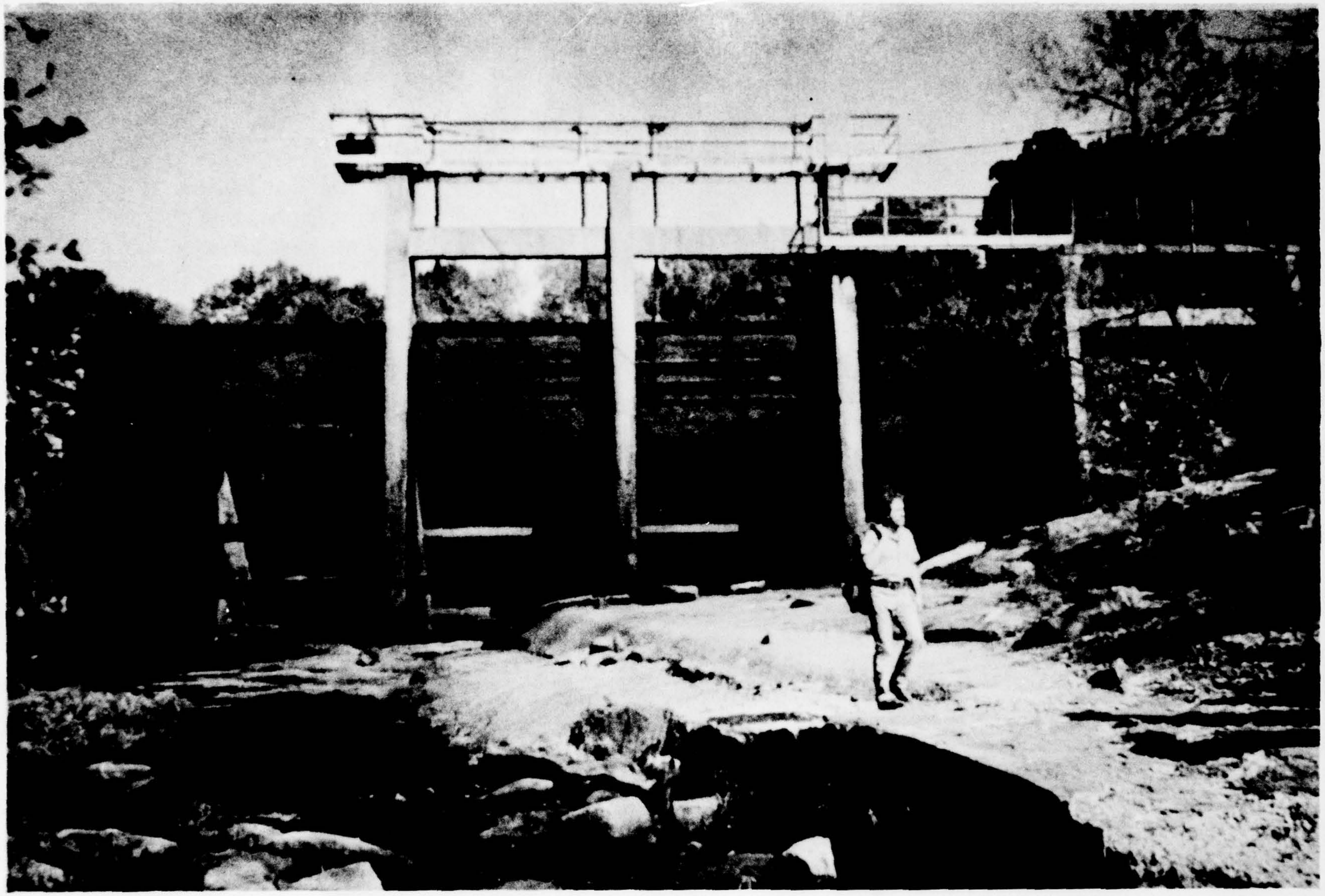
Approved: Original signed by
Douglas L. Haller

DOUGLAS L. HALLER
Colonel, Corps of Engineers
District Engineer

Recommended By:
Original signed by
ZANE M. GOODWIN

ZANG M. GOODWIN, P.E.
Chief, Engineering Division

Date: FEB 6 1979



OVERVIEW

SECTION 1

PROJECT INFORMATION

1.1 General:

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (Appendix VII, Reference 4). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Desciption:

1.2.1. Dam and Appurtendances: Falling Creek Filtation Plant Dam is a 207-foot long and 30-foot high (from steambed to top of non-overflow section, elev. 100.0) concrete buttressed dam. The buttresses are typically 17 feet 6 inches on centers, taper in cross section and founded on rock. A concrete slab of varying cross section rests on top of the buttresses and is inclined 45 degrees with the high point on the downstream side. (see photos, Appendix II). A concrete retaining wall separates the right abutment, composed of a 37-foot long earthen embankment with a concrete core, from the concrete portion of the dam.

The crest of the 155-foot long spillway is set at elev. 95.0 (m.s.l.). The uncontrolled spillway permits water to fall freely over the dam. Two 7-foot high by 16-foot wide steel sluce gates, top set at elev. 95.0, can be used to lower the reservoir to elevation 88.0. Lifting devices for the gates are supported on a high catwalk at elev. 107.0 over top the gates. The high catwalk is accessed from the left abutment across a lower catwalk set at elev. 100.

Water is drawn from the reservoir through a 16-inch diameter pipe (invert elev. 88.0) which connects to a 14 inch diameter supply pipe. A 24-inch drawdown pipe, designed to drain the reservoir, has never been used and assumed inoperable.

1.2.2 Location: Falling Creek Filtation Plant Dam is located on Falling Creek approximately 3 miles upstream of the Richmond-Petersburg Turnpike (I-95).

1.2.3. Size Classification: The dam is classified as an "intermediate" size structure based on storage potential (1511 acre-ft) and height (30 feet).

1.2.4. <u>Hazard Classification</u>: The dam is located upstream of a Filtation Plant and is given a "significant" hazard classification in accordance with guidelines contained in Section 2.1.2 of Reference 4, Appendix VI. The hazard classifications used to categorize dams are a function of location only and has nothing to do with its stability or probability of failure.

1.2.5. <u>Ownership</u>: Chesterfield County, Va.

1.2.6. <u>Purpose</u>: Water Supply

1.2.7. <u>Design and Construction History</u>: The dam was initially designed and constructed in 1952 to elevation 93.0 (spillway elev. [illegible]). The original designer was Perrow and Brockenbrough and the contractor was Thorington Construction Company. A 1956 addition, designed by R. Kenneth Weeks, Enginners, and constructed by English Construction Company, raised the dam to its present elevation of 100.0.

1.2.8. <u>Normal Operational Procedures</u>: Operation of the dam is automatic. The spillway does not require the use of the sluce gates and water rising above the crest of the spillway falls to the streambed below.

1.3 <u>Pertinent Data</u>:

1.3.1 <u>Drainage Areas</u>: The dam controls a drainage area of 53.24 square miles.

1.3.2 <u>Discharge at Dam Site</u>:

Maximum known flood since construction of dam - 5010 cfs in September 1960. Maximum flood of record - 7,270 cfs in July 1945. (gage 400 feet downstream).

Ungated spillway
Pool level at top of dam, elev 100.0 . . . . 5,571 c.f.s.

Gated spillway
Pool level at crest of ungated spillway,
elev. 95.0 . . . . 1,919 c.f.s.

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are shown in the following table:

Table 1.1 DAM AND RESERVOIR DATA

Item	Elevation feet m.s.l.	Reservoir			
			Capacity		
		Area acres	Acre feet	Watershed Inches	Length miles
Top of dam	100	144.6	1511	.53	3.3
Ungated spillway crest	95	91.8	920	.32	2.7
Gated spillway crest	88	59.6	390	.14	2.0
Streambed	68±	-	-	-	-

SECTION 2

ENGINEERING DATA

2.1 Design: Design drawings on both the original and the 1956 addition are on file with the owner, Chesterfield County, Department of Utilities. The plans include all necessary details to construct the dam. The 1956 drawings are marked "as-builts." (see appendix I). A set of structural design notes, for the 1956 addition, were found in the Engineers files. The notes contain a check of the orignal dam and a design of the addition. A full discussion of the notes appears in Section 6 - Dam Stability.

2.2. Construction: In addition to "as-built" plans, several concrete cylinder tests were found for the 1956 addition. The tests indicate the concrete used in the dam was well above 3,000 P.S.I. concrete. (See Appendix V)

2.3. Independent Phase I Report: In March 1978, the consulting firms of J. K. Timmons & Associates, Inc. and Schnabel Engineering, Assoc. performed a Phase I inspection at the request of the owner. The inspection, conducted in accordance with Corps of Engineers criteria for Phase I inspections, contains three major concerns regarding the safety of the dam; deterioration of joint sealant, deterioration of concrete buttress shelves and potential overflow on to the embankment during a hundred year storm. The owner plans to implement all remedial measures presented in the report.

2.4. Evaluation: The two sets of existing plans adequately describe the dam in detail. Evaluation of the design notes appears in Section 6. The March 1978, Phase I report is adequate and the general assessment and recommendations appear sufficient to correct the deficiencies reported.

SECTION 3

VISUAL INSPECTION

3.1 Foundation, Embankment, and Abutments:

The visual inspection of the foundation conditions was obscured by local ponding and debris. The findings outlined in the J. K. Timmons & Associates, Inc. March 1978 report were used as a guideline. The Corps was unable to locate the 4 seeps outlined in Section B, Geotechnical, Field Inspection. Possible reasons for missing the seeps are outlined in Appendix III, Field Observations, Concrete Dam, Foundations. However, an additional foundation seep was located at the downstream base of Buttress 6 and noted on Plate 3.

Large quantities of iron bacteria were found in local ponding underneath the spillway. The significance and origin of the bacteria is unclear.

The embankment and abutments showed no signs of cracking, movements, sloughing, or erosion. However, the right embankment was vegetated with several coniferous and deciduous trees.

3.2 Concrete Spillway, Slabs, Buttresses and Joints: The visual inspection of the concrete portion of the dam was hindered by dense vegetation along the non-overflow sections and water flowing down the face of the dam. The J. K. Timmons & Associates, Inc., March 1978 report was again used as a guideline. Very few cracks were observed in the underside of the slabs or buttresses. The concrete appeared in good condition. Seepage and deterioration around the slab seats on the buttresses were very noticable. Joints and joint sealent were almost non observable.

3.3 Evaluation: The visual inspection of the foundation revealed two significant conditions that warrant attention. First, the previously identified seeps, and second, the additional seep. These seeps should be marked in the field for easy identification. They should also be monitored and treated as specified in the Timmons report. Also, the origin and significance of the iron bacteria should be determined.

A less significant condition is the vegetation on the right embankment. It is not recommended that the trees be cut because of their extensive growth. It is suspected that killing the trees would cause decay and encourage piping.

The visual inspection confirmed the findings of the J. K. Timmons Phase I report concerning concrete portions of the structure. The remedial measures given in the report for these findings should be implemented.

SECTION 4

OPERATIONAL PROCEDURES

4.1. Procedures and Maintenance: Operating procedures are handled as the need arises. Debris removal is accomplished irregularly. The sluice gates and lifting mechanism have only been operated two or three times, the latest operation of the gates being in March 1978 for the independent Phase I report. At that time, only one of the gates was raised and closed. The 16-inch water supply and 24-inch drawdown valves remain in one position and are never operated. The valve operator is missing from the 24-inch valve.

4.2. Warning Systems: No warning system is maintained by Chesterfield County.

4.3. Evaluation: The dam does not require an elaborate operational and maintenance procedure. However, a plan of preventive maintenance for the sluice gates, lifting mechanism, and the valves should be developed and followed.

SECTION 5

HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: There are no original hydraulic or hydrologic design data available for the Falling Creek Filtration Plant Dam.

5.2 Hydrologic Records: Reservoir pool elevations are recorded once a month by personnel from the Falling Creek Filtration Plant, but not maintained at the plant. The U.S. Geological Survey has maintained flow records approximately 5 miles upstream of the dam on Falling Creek (drainage area 32.8 square miles) since August 1955. Another gage was located approximately 400 feet downstream of the dam between 1943 and 1964 (drainage area 54 square miles). Peak discharges recorded at these gages are shown in the following table:

TABLE 5.1
FALLING CREEK FLOOD RECORD 1/

GAGE	DRAINAGE AREA SQ.MI.	PERIOD OF RECORD
Near Chesterfield	32.8	1955 - Present
Near Drewrys Bluff	54.0	1942 - 1964

Date of Flood	Chesterfield	Drewrys Bluff
	Peak Discharges c.f.s.	
18 Jul 1945	—	7270
13 Aug 1955	—	3100
18 Aug 1955	2000	4700
12 Sep 1960	2500	5010
21 Oct 1961	1500	2780
6 Jan 1962	1450	2800
23 Jul 1969	1200	—
14 Jul 1975	1600	—

1/ Includes all recorded floods over 1000 cfs at Chesterfield and 2000 cfs at Drewrys Bluff gages.

5.3 Flood Experience: The maximum pool level sited by plant personnel was approximately at elevation 96.5± or 3.5 feet below the top of dam. Calculations indicated the peak discharge of 5010 cfs of 12 September 1960 would have caused a rise to within one foot of the top of dam. No records or visual sightings show that the dam has been overtopped.

5.4 Flood Potential: The Probable Maximum Flood (PMF), 1/2 PMF, and 100-year flood were developed and routed through the reservoir by use of the HEC-1DB computer program (Reference 1, Appendix VI) and appropriate unit hydrograph, precipitation, and storage-outflow data. Clark's T_c and R coefficients for the local drainage area were estimated from basin characteristics and actual rainfall and flood hydrograph records. The rainfall applied to the developed unit hydrograph was obtained from U. S. Weather Bureau Publications (Reference 2 and 3, Appendix VI). Losses were estimated at an initial loss of 0.80 inch and a constant loss thereafter of 0.05 inch/hour.

5.5 Reservoir Regulation: Releases through a 14-inch pipe supplies water to the Falling Creek Filtration Plant, which provides water to the County of Chesterfield. Under normal streamflow conditions, the reservoir pool level is approximately elevation 95. Excess streamflows are automatically passed over the ungated spillway. Two gates can be raised to lower the pool to elevation 88.0.

A storage curve was developed for the reservoir from known storage values, then extended above the top of dam by use of U. S. Geological Survey Quadrangle Maps. A rating curve with the gated spillway closed was developed for the ungated spillway and non-overflow section of the dam. For pool elevations above top of dam (elevation 100.0), the gates, when lifted, allow less flow to pass downstream. In routing hydrographs through the reservoir, it was assumed that the initial pool level was at the crest of the ungated spillway. Flow through the 14-inch water supply pipe was not considered in the routing. A tailwater rating curve was developed using a cross section of the channel immediately downstream of the dam.

5.6 Overtopping Potential: The probable rise of the reservoir and other pertinent information on reservoir performance is shown in the following table:

Table 5.2 RESERVOIR PERFORMANCE

Item	Normal Flow	Hydrograph		
		One Per-Cent 1/	½ PMF	PMF 2/
Peak flow, c.f.s.				
Inflow	53	14,189	26,252	52,517
Outflow	53	14,354	26,205	52,286
Max. elev., ft. msl	-	103.1	106.5	112.1
Ungated Spillway (El 95)				
Depth of flow, ft.	-	8.1	11.5	17.1
Duration, hours	-	65	108	111
Velocity, f.p.s.	-	9.4	11.2	13.6

Non-overflow section (El 100)				
Depth of flow, ft.	-	3.1	6.5	12.1
Duration, hours	-	15	21	30
Velocity, f.p.s.	-	4.7	6.8	9.2
Tailwater elevation, ft m.s.l.	68.5±	91	96.5	107.5

1/ The one percent exceedence frequency flood has one chance in 100 of being exceeded in any given year.
2/ The Probable Maximum Flood is an estimate of flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

5.7 Reservoir Emptying Potenial: The 24-inch gated outlet at elevation 70.0, if operable, is available for dewatering the reservoir. The gated spillway may be opened to lower the reservoir pool to elevation 88.0. With the reservoir pool at the crest of the ungated spillway, it would take 13 hours to lower the reservoir pool to the gated spillway crest (gates opened) and 8 days to lower reservoir pool to approximately elevation 82. An average flow of 53 cfs into the reservoir is assumed from 53 square miles of drainage area above the dam. Equilibrium, inflow equals outflow, will occur at elevation 80±.

5.8 Evaluation: This dam is given "intermediate" size and "significant" hazard classifications in which guidelines (appendix Vii, reference 4) recommended a spillway design flood of 1/2 pmf to pmf as being appropriate. It is considered that a spillway design flood of 1/2 pmf is more clearly related to the risk involved in this project and is therefore selected as the spillway design flood.

The spillway will pass only 21% of the recommended spillway design flood. The reservoir will rise 3.1 and 6.5 feet above the top of the dam in the one percent exceedence frequency flood and the recommended spillway design flood, respectfully.

The effect of future development on the hydrology is not reflected in conclusions presented herein.

SECTION 6

DAM STABILITY

6.1 Stability Analysis: A stability analysis performed during the time of the 1956 addition indicates the concrete portions of the dam are designed for a water level of elev. 95.0, spillway crest. Hydrology calculations give the 100-year flood at elev. 103±. The tailwater at the time of the 100-year flood has been calculated by the Corps of Engineers to be elev. 91. The left non-overflow section was designed for a maximum depth of water six feet below the top of dam (see sheet IV-1, Appendix IV). This gives a maximum design head at elevation 91.0 of three feet. The 100-year storm produces a design head at this elevation of approximately 12 feet. Similarily, the design notes use a seven-foot head to design the spillway slabs under the sluice gates (see sheet IV-13, Appendix IV). The 100-year storm produces a design head of twelve feet.

6.2 Foundation, Embankment, and Abutments: The Geotechnical section prepared by Schnabel Engineering Associates and presented in the Timmons report covers the foundation, embankment, and abutments. The report covers the regional geology, review of available design data, results of the March 1978 inspection and potential erosion due to overtopping. The report is on file with the owner.

6.3 Evaluation: The examples of rather large descrepancies in the concrete design indicate further analysis should be performed by the owner to determine the structural integrety of the dam.

SECTION 7

ASSESSMENT/REMEDIAL MEASURE

7.1 Dam Assesment: With the exception of deficiencies outlined in the J. K. Timmons & Associates, Inc. report, the dam, from a visual aspect, appears to be in good condition. The structural design notes do not appear to account for flood conditions in the design.

Based on hydrology calculations, the spillway is capable of passing 21% of the 1/2 P.M.F., the recommended spillway design flood appropriate to this dam. Therefore, the spillway is rated inadequate.

The spillway will pass only 21% of the recommended spillway design flood. The reservoir will rise 3.1 and 6.5 feet above the top of the dam in the one percent exceedence frequency flood and the recommended spillway design flood, respectfully.

The effect of future development on the hydrology is not reflected in conclusions presented herein.

7.2 Recommended Remedial Measures: The owner should immediately implement the following recommendations:

7.2.1 The remedial treatment as outline in the J. K. Timmons report should be accomplished.

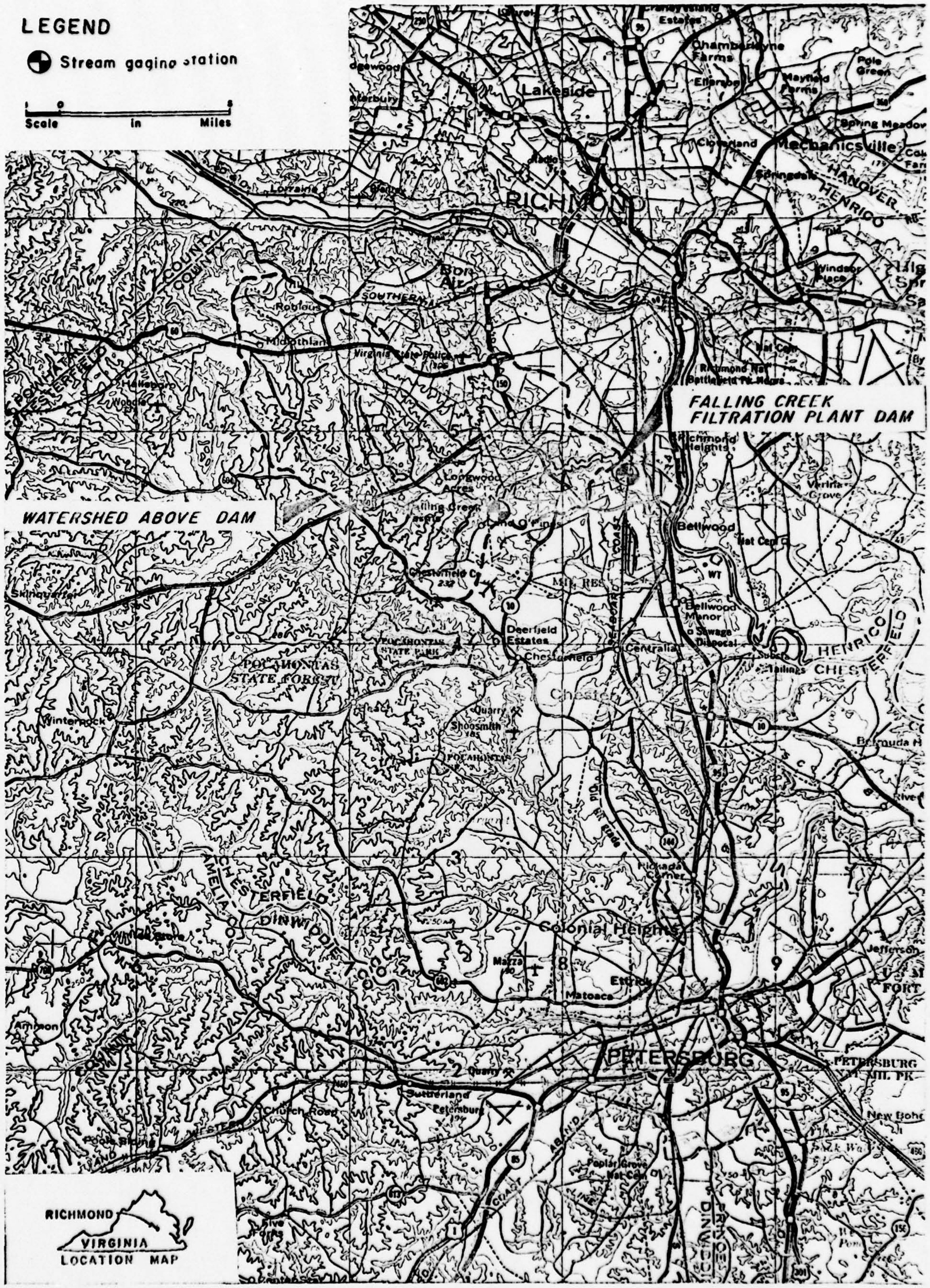
7.2.2. The owner, through his professional Engineers, should investigate the structural stability of the dam under all conditions up to and including the 1/2 P.M.F. Any resulting remedial measures should be undertaken to insure the safety of the dam.

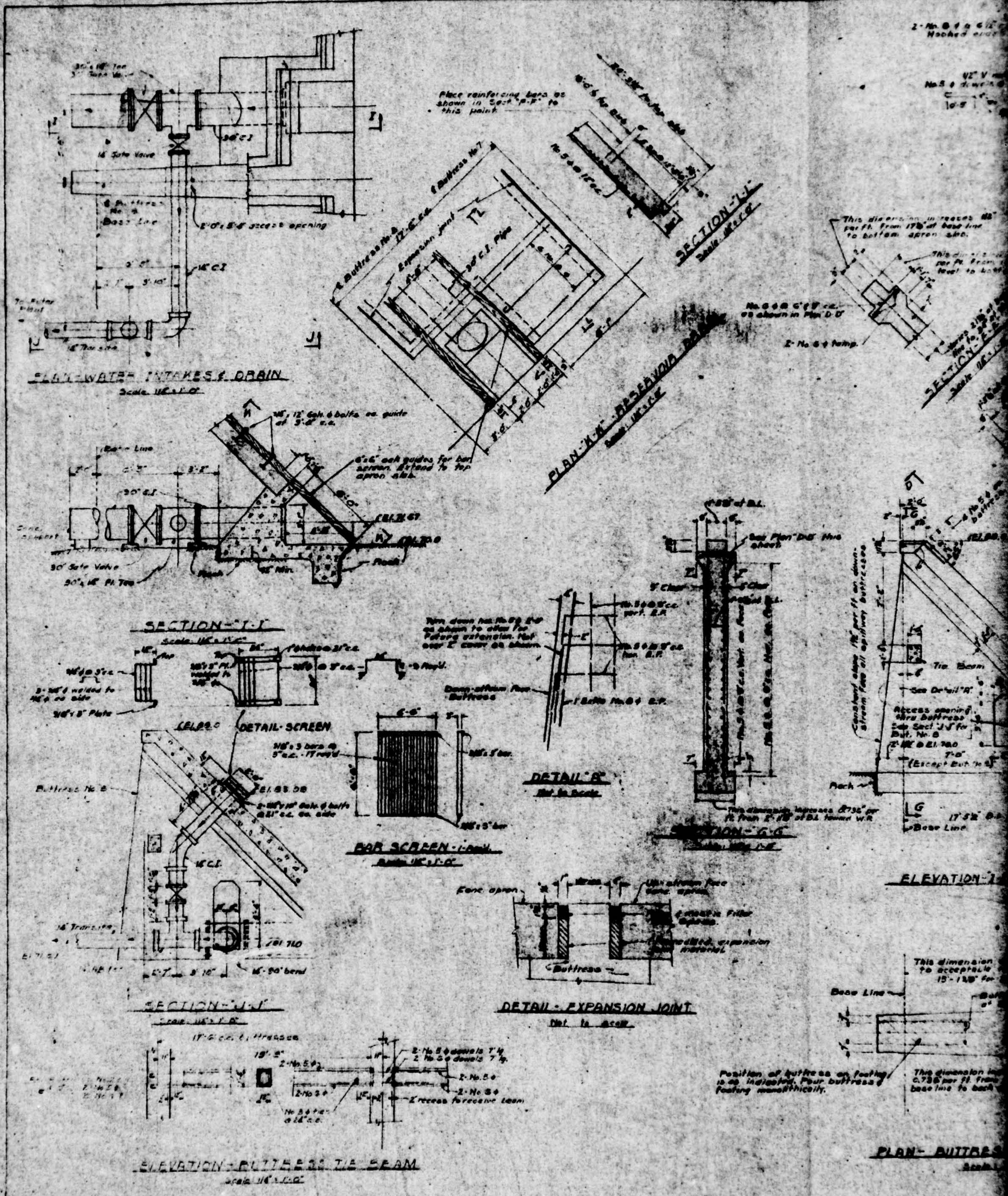
7.2.3 A plan of preventive maintenance for the sluice gates, lifting mechanism, and the valves should be developed and followed.

7.2.4 The seeps identified in the visual inspection and the J. K. Timmons report should be monitored and treated as specified in the report.

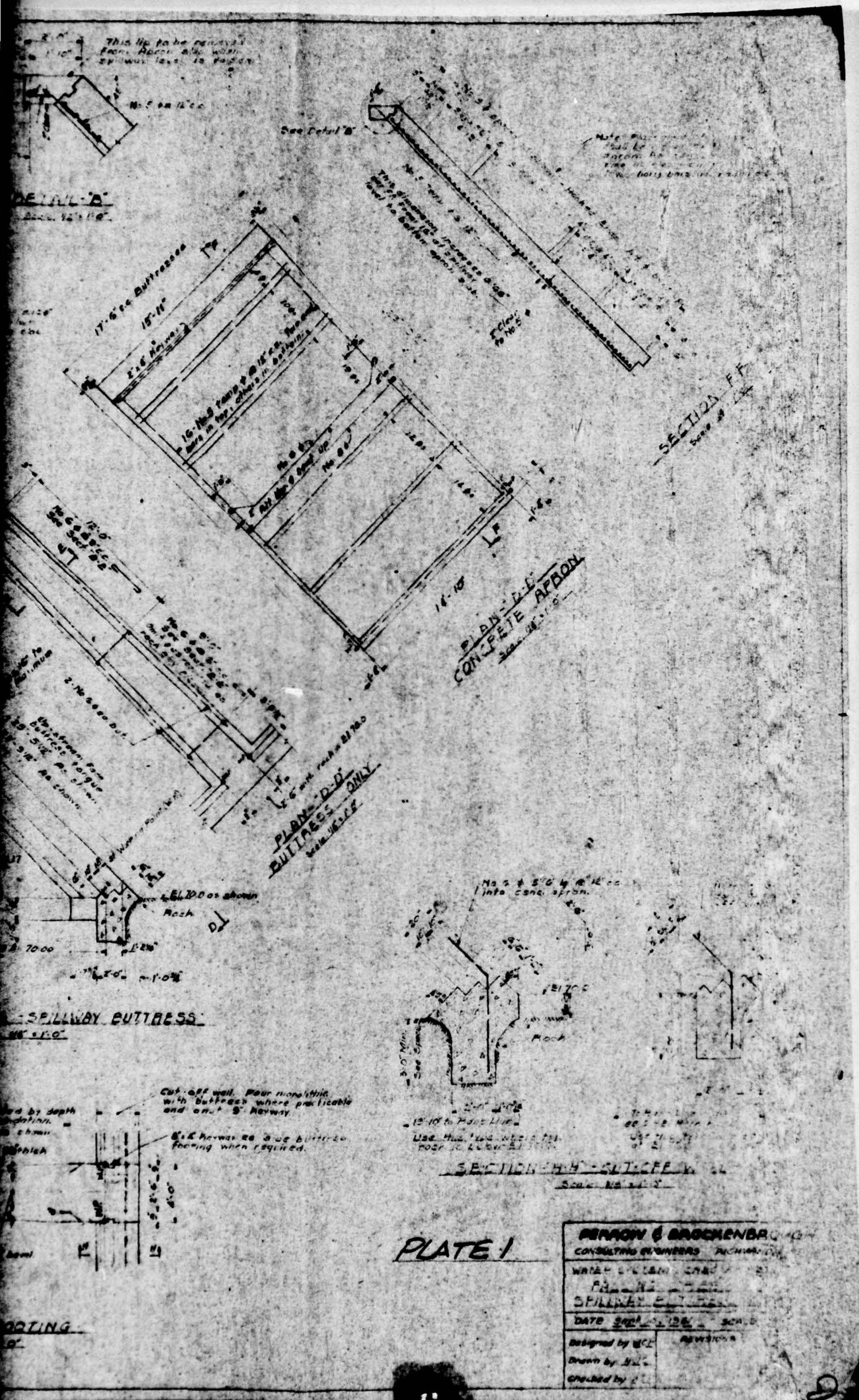
APPENDIX I

MAPS AND PLATES

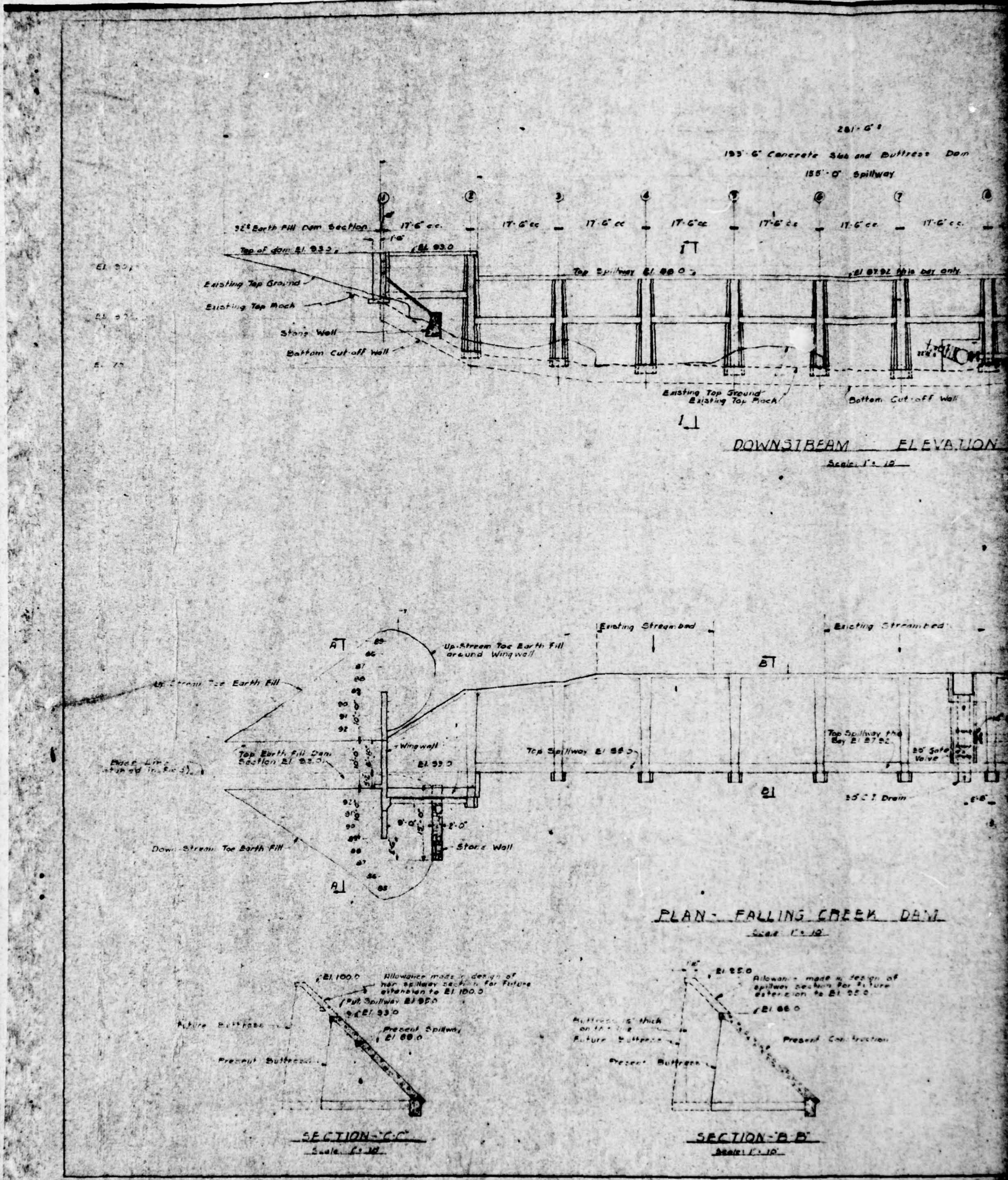

LEGEND
Stream gaging station
Scale in Miles
FALLING CREEK
FILTRATION PLANT DAM
WATERSHED ABOVE DAM
RICHMOND
VIRGINIA
LOCATION MAP
RICHMOND
Lakeside
Mechanicsville
HANOVER
HENRICO
Midlothian
Bellwood
Deerfield Estates
Centralia
Chester
CHESTERFIELD
DINWIDDIE
Colonial Heights
Matoaca
Ettrick
PETERSBURG
Sutherland
Church Road



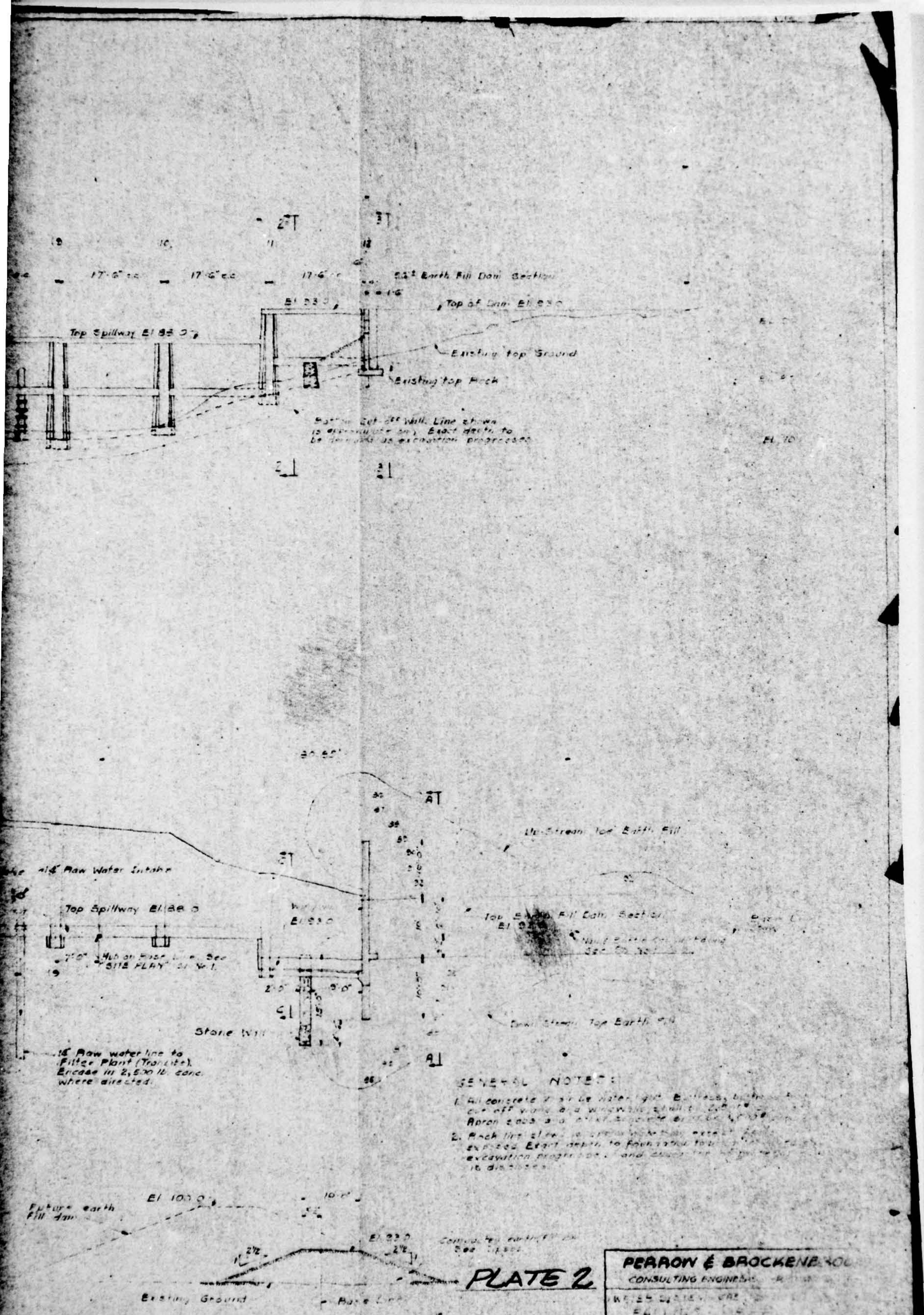
FLUSH-WATER INTAKES & DRAIN
Scale 1/8" = 1'-0"
16" Gate Valve
2'-0" x 5'-6" access opening
16" C.I.
PLAN "K-K" RESERVOIR DAM
SECTION "L-L"
Place reinforcing bars as shown in Sect. "F-F" to this point
Expansion joint
30" C.I. Pipe
SECTION "I-I"
Scale 1/8" = 1'-0"
30" Gate Valve
Rock
DETAIL-SCREEN
Buttress No. 8
16" C.I.
16" Transite
16" - 90° bend
SECTION "J-J"
BAR SCREEN - 1 Req'd.
DETAIL "A"
Not to Scale
SECTION "G-G"
DETAIL - EXPANSION JOINT
Not to scale
Buttress
Position of buttress on footing is as indicated. Pour buttress & footing monolithically.
ELEVATION - BUTTRESS TIE BEAM
ELEVATION
PLAN - BUTTRESS
Tie Beam
Access opening thru buttress
Base Line



DETAIL "B"
See Detail "B"
17'-6" c.c. Buttresses
15'-11"
16-10
PLAN "D-D" CONCRETE APRON
PLAN "D-D" BUTTRESS ONLY
SECTION F-F
SPILLWAY BUTTRESS
Rock
Cut-off wall. Pour monolithic with buttress where practicable and omit 5" keyway.
SECTION H-H - CUT-OFF WALL
FOOTING
PLATE 1
PERROW & BROCKENBROUGH
CONSULTING ENGINEERS
SPILLWAY BUTTRESS
DATE
REVISIONS
Designed by
Drawn by
Checked by

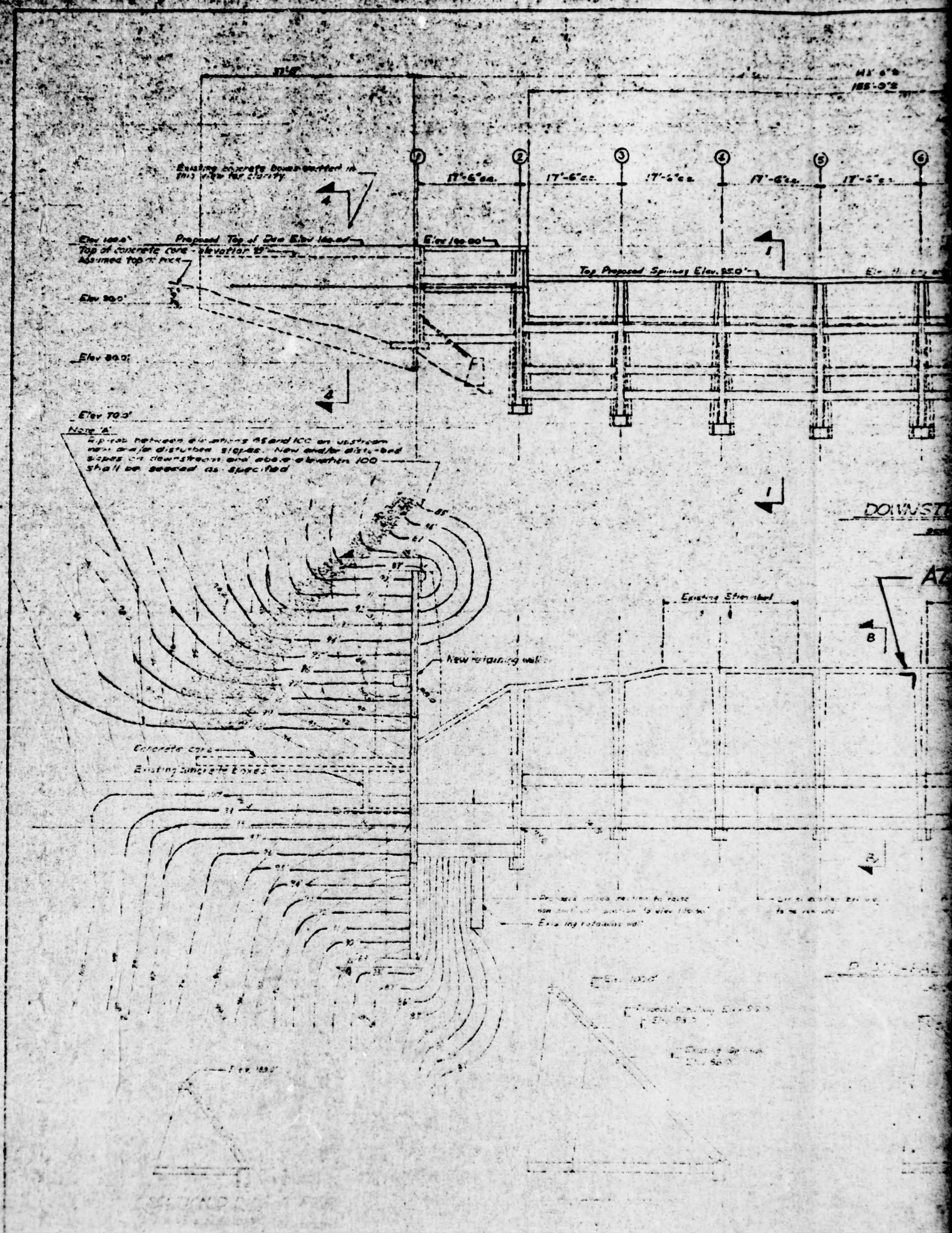


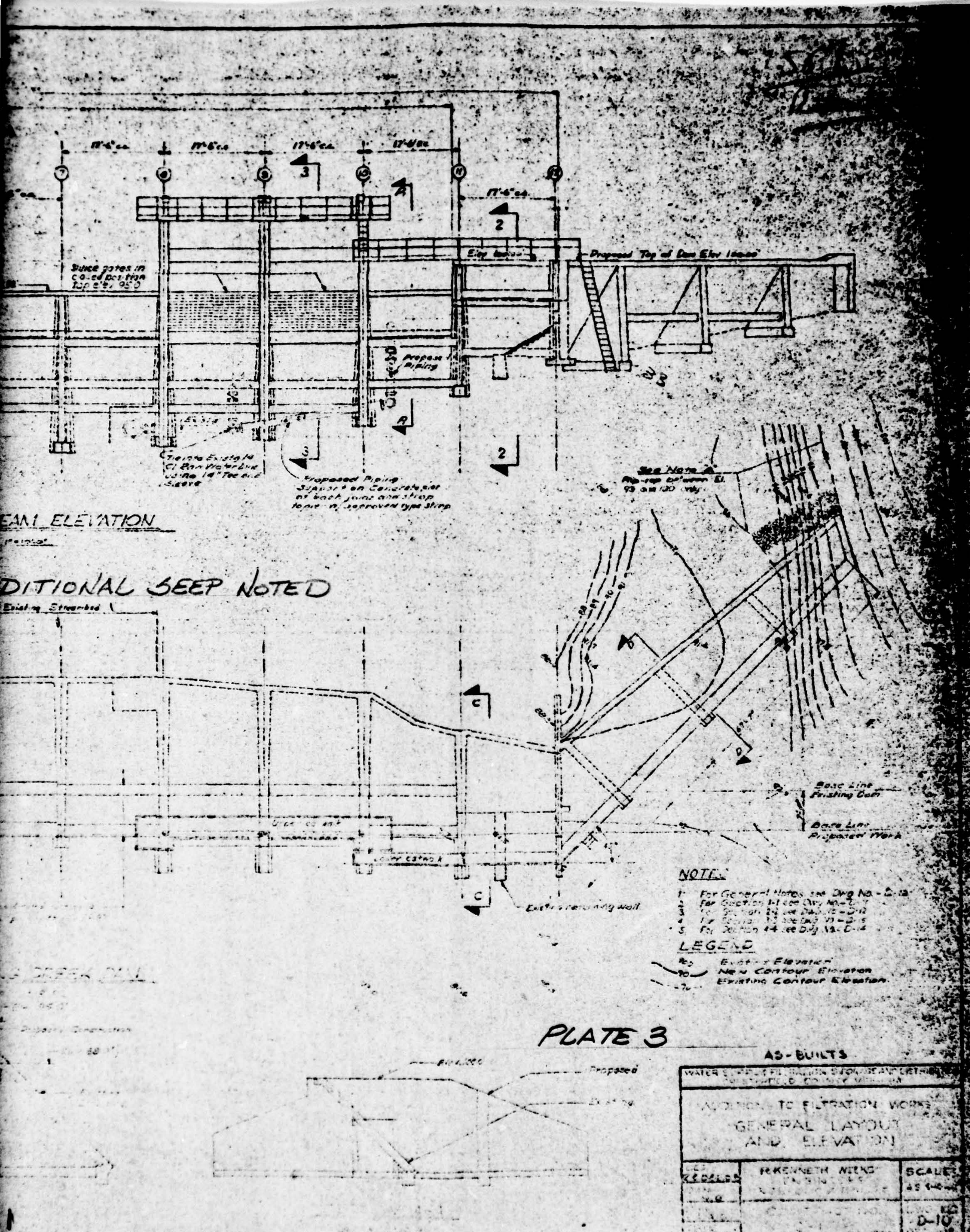
281'-6"
193'-6" Concrete Slab and Buttress Dam
155'-0" Spillway
92' Earth Fill Dam Section
17'-6" c.c.
Top of dam El. 93.0
El. 93.0
Top Spillway El. 88.0
El. 87.92 this bay only
Existing Top Ground
Existing Top Rock
Stone Wall
Bottom Cut-off Wall
Existing Top Ground
Existing Top Rock
Bottom Cut-off Wall
DOWNSTREAM ELEVATION
Scale: 1" = 10'
Existing Streambed
Existing Streambed
Up-Stream Toe Earth Fill around Wingwall
Up-Stream Toe Earth Fill
Wingwall
Top Earth Fill Dam Section El. 93.0
El. 93.0
Top Spillway El. 88.0
Top Spillway this Bay El. 87.92
Safety Valve
C.I. Drain
Down-Stream Toe Earth Fill
Stone Wall
PLAN - FALLING CREEK DAM
Scale: 1" = 10'
El. 100.0
Allowance made in design of new spillway section for future extension to El. 100.0
Fut. Spillway El. 95.0
El. 93.0
Future Buttress
Present Spillway El. 88.0
Present Buttress
SECTION "C-C"
Scale: 1" = 10'
El. 95.0
Allowance made in design of spillway section for future extension to El. 95.0
El. 88.0
Future Buttress
Present Construction
Present Buttress
SECTION "B-B"
Scale: 1" = 10'



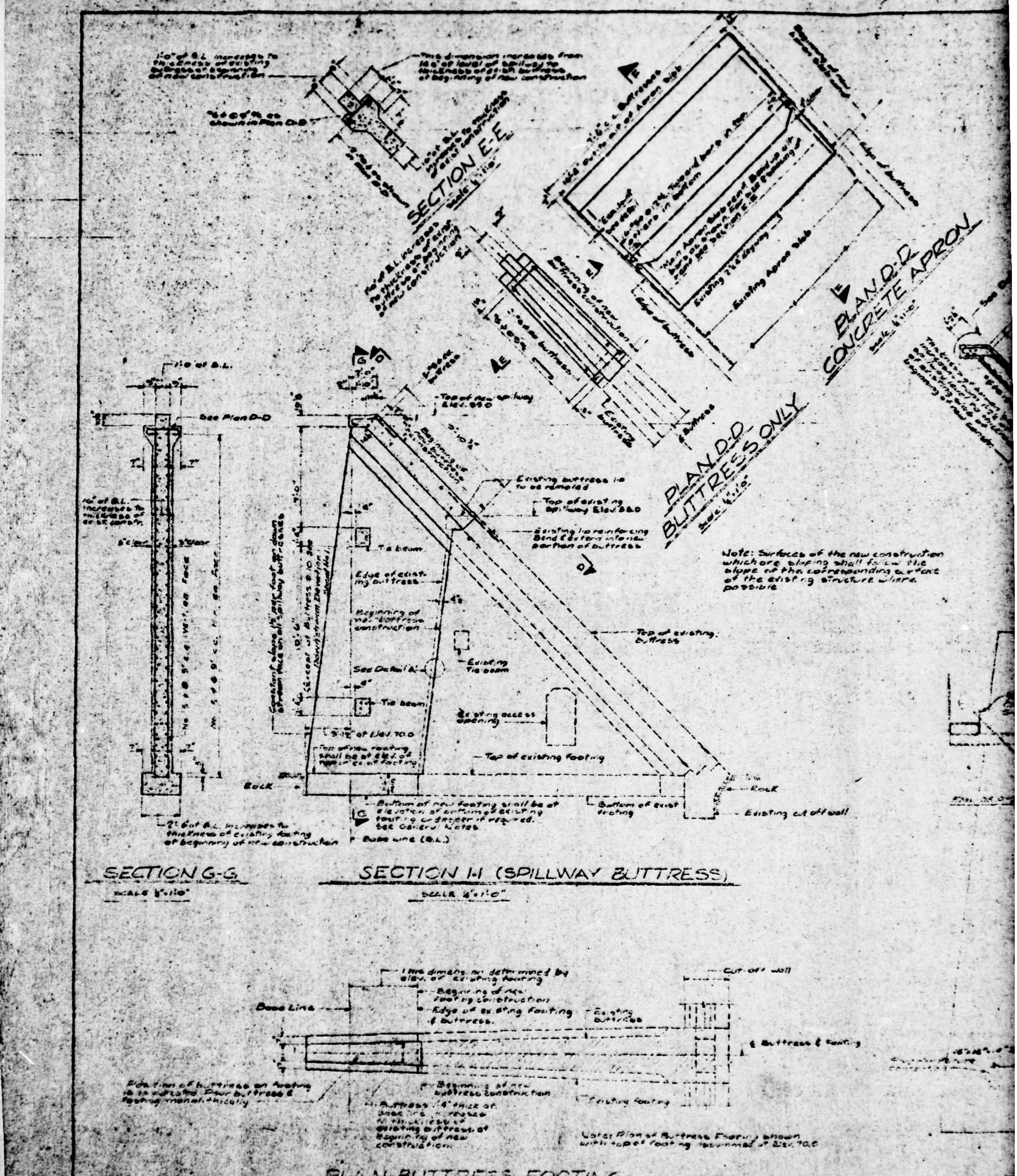
Top Spillway El 88.0
Existing top Ground
Existing top Rock
Top of Dam El 93.0
Raw Water Intake
Top Spillway El 88.0
Stone Wall
16" Raw water line to Filter Plant (Transite). Encase in 2,500 lb. conc. where directed.
Up Stream Toe Earth Fill
Down Stream Toe Earth Fill
GENERAL NOTES:
Future earth Fill dam
El 100.0
El 93.0
Existing Ground
Base Line
SECTION "A-A"
PERROW & BROCKENBROUGH
CONSULTING ENGINEERS
GENERAL
DATE
Designed by
Drawn by

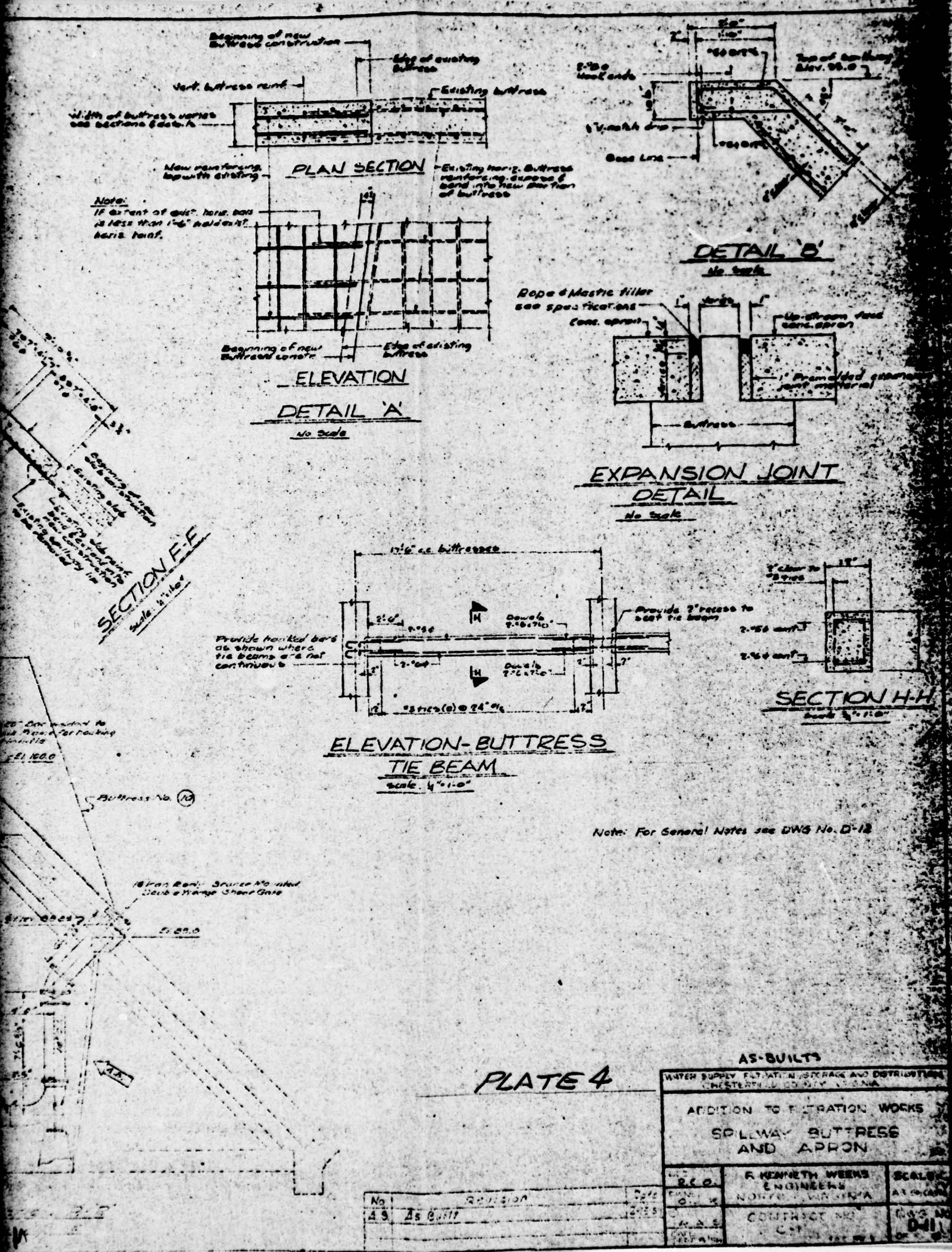
PLATE 2


Existing concrete boxes omitted in this view for clarity
17'-6" c.c.
Elev 100.0'
Proposed Top of Dam Elev 100.00'
Elev 100.00'
Top of concrete core - elevation 99'
Assumed top of rock
Elev 90.0'
Top Proposed Spillway Elev. 95.0'
Elev 80.0'
Elev 70.0'
Note 'A'
Rip-rap between elevations 95 and 100 on upstream new and/or disturbed slopes. New and/or disturbed slopes on downstream end above elevation 100 shall be seeded as specified
Existing Streambed
New retaining wall
Concrete core
Existing concrete boxes
Existing retaining wall



Sluice gates in closed position
Proposed "Top of Dam Elev 1060"
Proposed Piping
Proposed Piping Support on Concrete pier at each joint and strap to pier w/ approved type strap
EAM ELEVATION
DITIONAL SEEP NOTED
Existing Streambed
See Note A
Rip-rap between El. 93 and 120 only
Base Line Existing Dam
Base Line Proposed Work
Existing retaining wall
NOTES
LEGEND
Existing Elevation
New Contour Elevation
Existing Contour Elevation
Proposed
Existing
PLATE 3
AS-BUILTS
ADDITIONS TO FILTRATION WORKS
GENERAL LAYOUT AND ELEVATION
R. KENNETH WEEKS ENGINEERS
SCALE
D-10
2


SECTION E-E
PLAN D-D CONCRETE APRON
PLAN D-D BUTTRESS ONLY
Note: Surfaces of the new construction which are sloping shall follow the slope of the corresponding surface of the existing structure where possible
Top of existing buttress
Existing access opening
Top of existing footing
Rock
Existing cut off wall
Bottom of exist footing
Base Line (B.L.)
SECTION G-G
SECTION I-I (SPILLWAY BUTTRESS)
Base Line
Cut-off wall
Existing footing
Note: Plan of Buttress Footing shown with top of footing assumed at Elev. 70.0
PLAN-BUTTRESS FOOTING

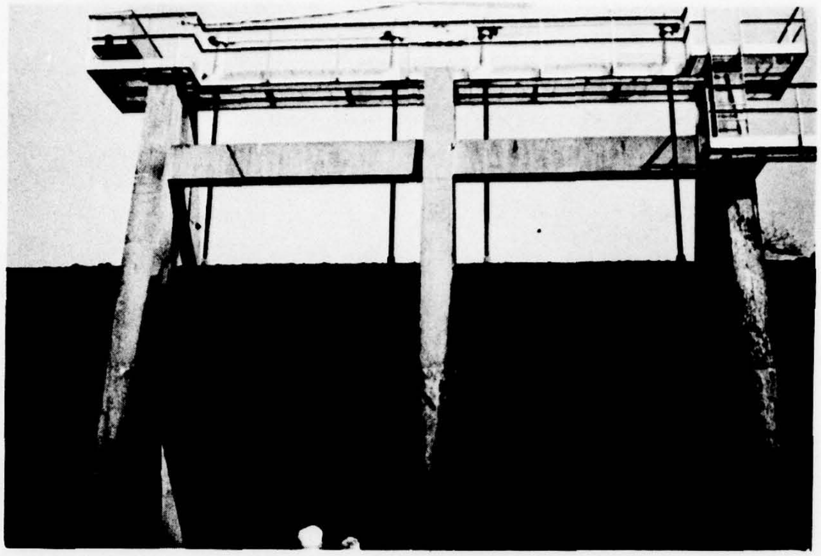

PLAN SECTION
Note:
If extent of exist. horiz. bars is less than 1'-6" weld exist. horiz. reinf.
ELEVATION
DETAIL 'A'
No Scale
DETAIL 'B'
No Scale
Rope & Mastic filler see specifications
EXPANSION JOINT DETAIL
No Scale
SECTION F-F
ELEVATION - BUTTRESS TIE BEAM
SECTION H-H
Note: For General Notes see DWG No. D-12
PLATE 4
AS-BUILTS
WATER SUPPLY FILTRATION STORAGE AND DISTRIBUTION
ADDITION TO FILTRATION WORKS
SPILLWAY BUTTRESS AND APRON
F. KENNETH WEEKS ENGINEERS
CONTRACT NO.
As Built

APPENDIX II

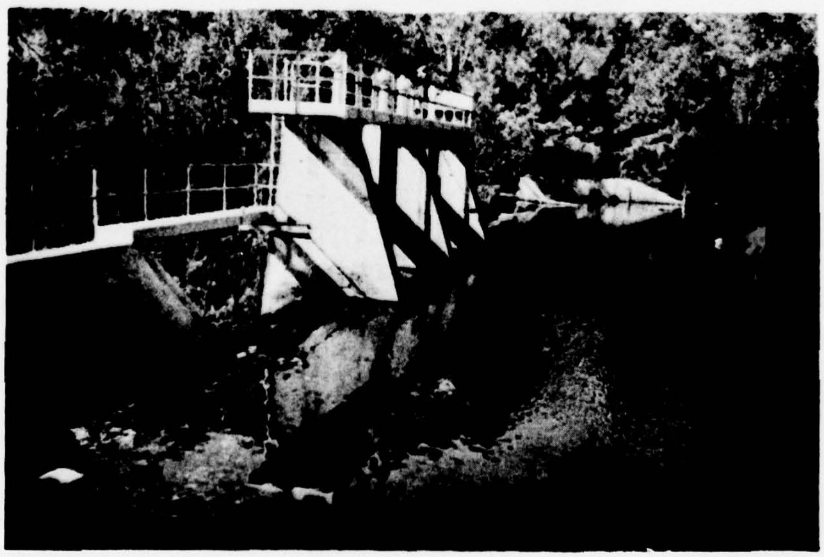
PHOTOGRAPHS

STEEL GATE LIFT ASSEMBLY

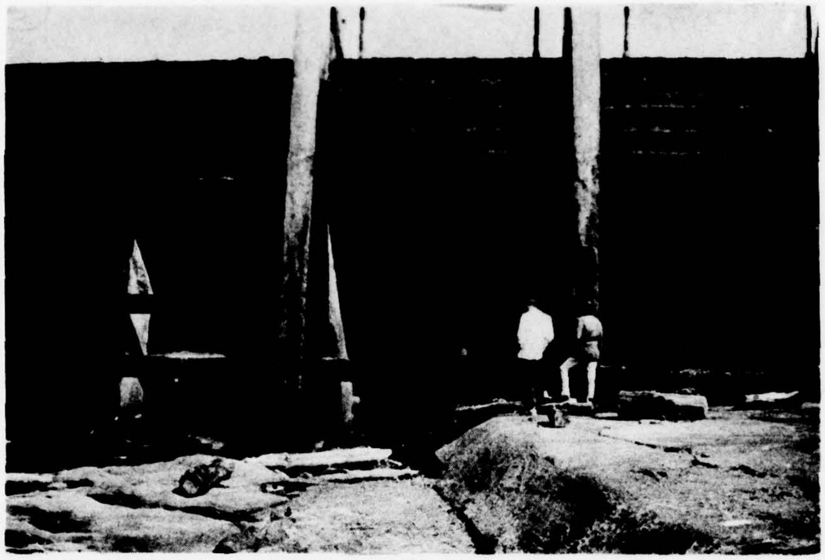
VIEW OF DAM FROM RIGHT ABUTMENT



STEEL GATES AND LIFT ASSEMBLY



UPSTREAM VIEW OF LIFT ASSEMBLY



14 INCH WATER SUPPLY PIPE AND CONCRETE APRON

DOWNSTREAM CHANNEL

UNDERSIDE OF SLAB AT GATES

24 INCH CAST IRON DRAIN AND VALVE

APPENDIX III

FIELD OBSERVATIONS

Check List
Visual Inspection
Phase 1

Name Dam Falling Creek Filtation Plant **County** Chesterfield **State** Virginia **Coordinates** Latitude 3718.1
Longitude 7950.2

Date(s) Inspection 1 Nov 1978 **Weather** Fair **Temperature** 70°

Pool Elevation at Time of Inspection 95' m.s.l. **Tailwater at Time of Inspection** (Steambed + .5') 68.5 m.s.l.

Inspection Personnel:

State Water Control Board:	Jack Hyden Chenchaya Bathala
Chesterfield County:	Robert A. Painter Craig S. Bryant Donald E. Addison
Corps of Engineers:	David A. Pezza David Dougan Jim Robinson William A. Sorrentino Jeffery C. Irving (Recorder)

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEEPAGE OR LEAKAGE	Seepage is apparent along the top of several buttresses. Water flowing over the spillway hinders a true evaluation of the seeps.	This condition is written up in J. K. Timmons Phase I Report and remedial measures are prescribed.
STRUCTURE TO ABUTMENT/EMBANKMENT FUNCTIONS	Very Vegetated. Could not tell very much.	
DRAINS	NONE	
WATER PASSAGES	2 1-24" At base which empties dam (closed) 1-16" Thru dam for water supply (open), The valves are not operated	Preventive maintenance should be provided for all valves.
FOUNDATION	Reference Timmons & Assoc.'s Report, Section B, Geotechnical, Drawing No. 4. The inspector is unable to locate the four seeps identified on Drawing No. 4. The inspection of Seeps 1 & 2 is hampered by local ponding clouded with iron bacteria and debris. The exact location of Seep 3 is not clear, but no seepage in the general area is apparent. It is suspected that Seep 4 is submerged. An additional seep is apparent at the base of the right (West) downstream junction of the spillway and Buttress No. 6. The flow is easily observable and appears to come from beneath the dam. The amount and rate are difficult to estimate and local ponding is clouded with iron bacteria.	

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Minimal	See J. K. Timmons Report
STRUCTURAL CRACKING	Minimal	See J. K. Timmons Report
VERTICAL AND HORIZONTAL ALIGNMENT	N/A	
MONOLITH JOINTS	N/A	
CONSTRUCTION JOINTS	NONE OBSERVED	See J. K. Timmons Report

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	There are no signs of cracks.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	There are no signs of movement or cracking.	
SLOUGHING OR EROSION	There are no signs of sloughing or erosion. The right embankment is vegetated with several coniferous and deciduous trees several inches in diameter. There are rock outcrops on the left abutment.	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	There are no apparent deviations from available drawings.	
RIPRAP FAILURES	Portions of the downstream area and the area at the toe of the embankment appear to be ripraped. The riprap in the downstream area is disheveled. The riprap at the toe of the embankment appears to be intact	
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	There are no signs of deterioration of any nature	
STAFF GAGE AND RECORDER	Staff gage located on left non-flow section.	
DRAINS	Available drawings do not show drains. No drains are apparent in the field.	

OUTLETS WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	N/A	
INTAKE STRUCTURE	Submerged unable to check (16" water supply reduces to 14" on d/s face)	
OUTLET STRUCTURE	14" water supply pipe to filtration plant buried beyond toe of dam.	Note: Wheel is available if needed at another location
OUTLET CHANNEL	Large rocks along streambed 20-30 ft. wide with 30°-35° slopes. Trees and shrubs Road Bridge approx. 100 yds. d/s	
EMERGENCY GATE	2 gates 7'x16' closed-opened twice in 15 years only for maintenance and Timmons Phase I 24" drawdown pipe (no wheel). Never been used.	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	155' wide Vertical downstream face 2' wide crest great slope upstream 70° large tree extends over right side of spillway	
APPROACH CHANNEL	RESERVOIR 2.7 miles long (narrow)	
DISCHARGE CHANNEL	Vertical drop to a pool formed by large rocks and boulders some concrete work noted in left portion (apron) of d/s channel at toe of dam. It slopes to stream center, protects water supply line against erosion (channel)	Remove dead tree limbs that have passed over the spillway.
BRIDGE AND PIERS	Gate lift assembly (steel) catwalk and motors appear in good shape. Hand rail noted along top of dam.	

GATED SPILLWAY

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	Unobserved	
APPROACH CHANNEL	N/A	
DISCHARGE CHANNEL	Vertical discharge to stream.	
BRIDGE AND PIERS	Rises above top of dam. Previous page	
GATES AND OPERATION EQUIPMENT	Forms ungated spillway when gate is closed operated only twice. Unlikely to be operated during storm.	

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	1'to 8' vertical from water line to elevation where road borders reservoir. Trees and shrubs line shore. Some small docks for fishing visible.	
SEDIMENTATION	None observed but 2 trees appeared to rest vertically near dam face so that the top branches can be seen.	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Some dead tree limbs just below toe of dam Bridge located 100 yds. d/s will cause backwater effect	
SLOPES	30^{o}-40^{o} slopes (wooded)	
APPROXIMATE NO. OF HOMES AND POPULATION	Filtration Plant 2-6 people. Moderately traveled road along reservoir and bridge below dam	

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

ITEM	REMARKS
PLAN OF DAM	Two sets of plans, original done in 1952 and addition done in 1956.
CONSTRUCTION HISTORY	Original dam designed in 1952 to spillway elevation of 88.0 by Perrow & Brockenbrough. Addition designed in 1956 by R. Kenneth Weeks, Engineer.
TYPICAL SECTIONS OF DAM	See Plans
HYDROLOGIC/HYDROLIC DATA	None exists from either design. J. K. Timmons did Phase I Report in 1977 and worked-up Hydrology
DESIGN REPORTS	Not Found
GEOLOGY REPORTS	Not Found
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	A set of structural design notes gives check of existing structure and design of addition.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	Only some concrete test cylinder results remain from the addition.
POST-CONSTRUCTION SURVEYS OF DAM	NONE
BORROW SOURCES	NOT KNOWN
SPILLWAY PLAN	See Plans
OPERATING EQUIPMENT PLANS & DETAILS	NONE UNKNOWN

ITEM	REMARKS
MONITORING SYSTEMS	NONE
MODIFICATIONS	N/A
HIGH POOL RECORDS	NONE
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	J. K. Timmons Phase I Report - March 1978
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	NONE
MAINTENANCE OPERATION RECORDS	NONE

CHECK LIST
HYDROLOLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: Slightly sloped, vegitated, undeveloped

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 95.0

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): NONE

ELEVATION MAXIMUM DESIGN POOL: 95.0 (Should be 1/2 P.M.F. elev. 106.5)

ELEVATION TOP DAM: 100.0

CREST:

a. Elevation 95.0
b. Type Concrete - Sharp Crested
c. Width 1'-0"
d. Length 155'
e. Location Spillover Approx. Center Dam
f. Number and Type of Gates 2 Steel Sluice Gates

OUTLET WORKS:

a. Type 16" pipe
b. Location Near Left Abutment
c. Entrance inverts 88.0
d. Exist inverts 88.0
e. Emergency draindown facilities 24" pipe

HYDROMETEOROLOGICAL GAGES: NONE

a. Type
b. Location
c. Records

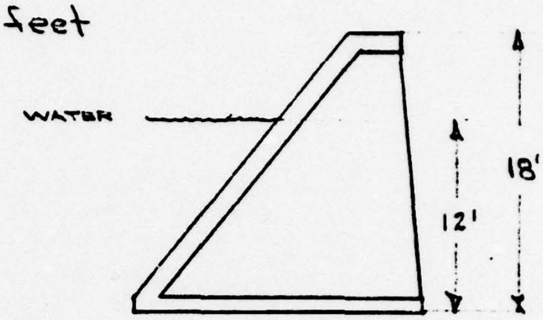
MAXIMUM NON-DAMAGING DISCHARGE: EL. 97.+ -

APPENDIX IV

STRUCTURAL CALCULATIONS

Design of inclined slab

Maximum depth of water is 12 feet

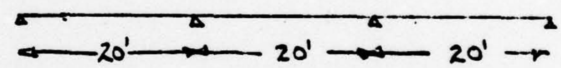


First examine a one foot strip along the bottom

Water pressure

$12 \times .0624 = .748\ K/_{ft^2}$

uniform load $.748^{K}/ft$



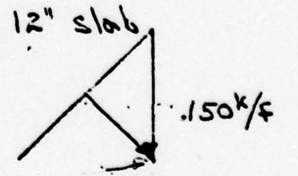
$\frac{wl^2}{14} = \frac{.748 \times 20 \times 20}{14} = 21.4$ f-K

$\frac{wl^2}{10} = \frac{.748 \times 20 \times 20}{10} = 29.9$ f-K

$\frac{wl^2}{16} = \frac{.748 \times 20 \times 20}{16} = 18.7$ f-K

Note — weight of concrete can be ignored in bottom strip since it is resting on rock.

Moment due to weight of concrete - assuming 12" slab



$\frac{.150}{1.41} = .106^{k}/_{f}$

$\frac{wl^2}{14} = \frac{.106 \times 20 \times 20}{14} = 3.03$ f-K

$\frac{wl^2}{10} = \frac{.106 \times 20 \times 20}{10} = 4.24$ f-K

$\frac{wl^2}{16} = \frac{.106 \times 20 \times 20}{16} = 2.65$ f-K

Examine strip 3 feet from bottom 85
Moments due to water are proportional to depth
Depth is 9 feet

Water + concrete

$\frac{9}{12} \times 21.4 = 16.1$, $16.1 + 3.03 = 19.1$ f-K

$\frac{9}{12} \times 29.9 = 22.4$, $22.4 + 4.24 = 26.6$ f-K

$\frac{9}{12} \times 18.7 = 14.1$, $14.1 + 2.65 = 16.8$ f-K

Examine strip 6 feet from bottom 88
Depth is 6 feet

$\frac{6}{12} \times 21.4 = 10.7$, $10.7 + 3.03 = 13.7$ f-K

$\frac{6}{12} \times 29.9 = 14.9$, $14.9 + 4.24 = 19.1$ f-K

$\frac{6}{12} \times 18.7 = 9.3$, $9.3 + 2.65 = 12.0$ f-K

Examine strip 9 feet from bottom 91
Depth is 3 feet

$\frac{3}{12} \times 21.4 = 5.35$, $5.35 + 3.03 = 8.38$ f-K

$\frac{3}{12} \times 29.9 = 7.48$, $7.48 + 4.24 = 11.7$ f-K

$\frac{3}{12} \times 18.7 = 4.67$, $4.67 + 2.65 = 7.32$ f-K

Examine strip at surface of water
Only moment is from weight of concrete

3.03 f-K
4.24 f-K
2.65 f-K

Steel design at bottom

Outer panel
$M = 21.4$ f-K $\quad A_s = \frac{M}{ad} = \frac{21.4}{1.44 \times 12} = 1.24\,\square''$ | bottom steel #8@$7\frac{1}{2}''$ = 1.26 □"
$a = 1.44$

In front of buttress
$M = 29.9$ f-K $\quad A_s = \frac{M}{ad} = \frac{29.9}{1.44 \times 12} = 1.73\,\square''$ | top steel #10@8" = 1.91 □"
$a = 1.44$

Inner panel
$M = 18.7$ f-K $\quad A_s = \frac{M}{ad} = \frac{13.7}{1.44 \times 12} = 1.08\,\square''$ | bottom steel #8@8" = 1.19 □"

Steel design 3' from bottom

Outer panel
$M = 19.1$ fK $\quad A_s = \frac{M}{ad} = \frac{19.1}{1.44 \times 12} = 1.1\,\square''$ | bottom steel #8@8" = 1.19 □"
$a = 1.44$

Front of buttress
$M = 26.6$ f-K $\quad A_s = \frac{M}{ad} = \frac{26.6}{1.44 \times 12} = 1.54\,\square''$ | top steel #10@9" = 1.69 □"
$a = 1.44$

Inner panel
$M = 16.8$ f-K $\quad A_s = \frac{M}{ad} = \frac{16.8}{144 \times 12} = 0.97\,\square''$ | bottom steel #8@9" = 1.05 □"
$a = 1.44$

Steel design 6' from bottom

Outer panel
$M = 13.7$ f-K $\quad A_s = \frac{M}{ad} = \frac{13.7}{1.44 \times 12} = 0.79\,\square''$ | bottom steel #7@9 = 0.80 □"
$a = 1.44$

Front of buttress
$M = 19.1$ f-K $\quad A_s = \frac{M}{ad} = \frac{19.1}{1.44 \times 12} = 1.1\,\square''$ | top steel #8@8" = 1.19 □"
$a = 1.44$

Inner panel
$M = 12.0$ f-K $\quad A_s = \frac{M}{ad} = \frac{12}{1.44 \times 12} = 0.69\,\square''$ | #7@10" = 0.72 □"
$a = 1.44$

Steel design 9 feet from bottom

Outer panel

$M = 8.38$ f-k $\quad A_s = \frac{M}{ad} = \frac{8.38}{1.44 \times 12} = 0.49 \square''$ | bottom steel #6 @ 10" = 0.53□"

$a = 1.44$

Front of buttress

$M = 11.7$ f-k $\quad A_s = \frac{M}{ad} = \frac{11.7}{1.44 \times 12} = 0.68 \square''$ | top steel #7 @ 10" = 0.72□"

$a = 1.44$

inner panel

$M = 7.32$ f-k $\quad A_s = \frac{M}{ad} = \frac{7.32}{1.44 \times 12} = 0.42 \square''$ | bottom steel #6 @ 12" = 0.44□"

$a = 1.44$

steel design at surface of water

outer panel

$M = 3.03$ f-k $\quad A_s = \frac{M}{ad} = \frac{3.03}{1.44 \times 12} = 0.18 \square''$ | bottom steel #5 @ 12" = 0.31□"

$a = 1.44$

Front of Buttress

$M = 4.24$ f-k $\quad A_s = \frac{M}{ad} = \frac{4.24}{1.44 \times 12} = 0.25 \square''$ | top steel #5 @ 12" = 0.31□"

$a = 1.44$

Inner panel

$M = 2.65$ f-k $\quad A_s = \frac{M}{ad} = \frac{2.65}{1.44 \times 12} = 0.15 \square''$ | bottom steel #5 @ 12" = 0.31□"

$a = 1.44$

Temperature steel - (minimum A_s allowed)

$.002 \times 12 \times 13 = 0.312 \square''$/ft of width

use #5 @ 12" o.c. $- A_s = 0.31 \square''$/ft

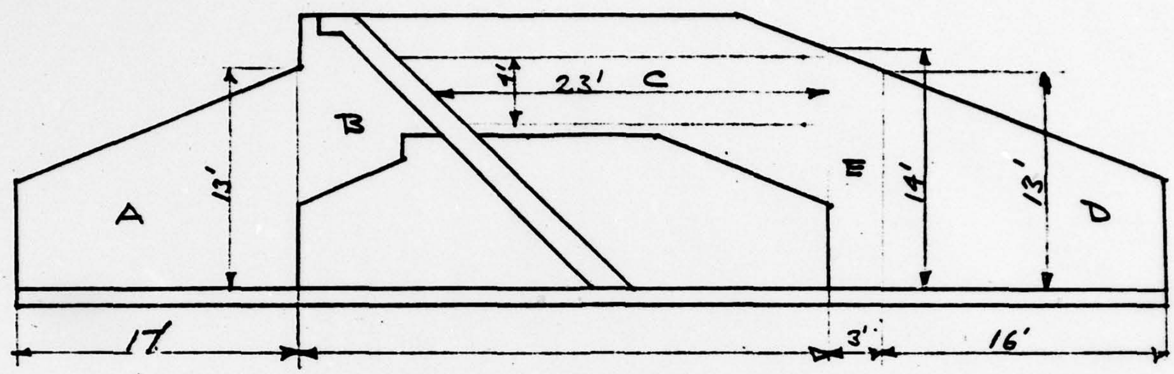
Comm. No. N-369 | **FRAIOLI-BLUM-YESSELMAN. CONSULTING ENGINEERS** | Sheet No. 5/13

fs 20,000 | Archt. | Date 9/24/56

fc 1,250 | Project | Subject | Des. By EDP

Design of retaining wall

Thickness of existing wall is 14"



Sections A & D are designed as conventional retaining walls. Section B is reinforced as a continuation of A. Section C is designed as a horizontal beam supported by the end of the dam slab and section E. Section E is designed as a retaining wall carrying the earth behind it and the reaction from the end of section 'C'.

Section 'C'

Span = 23'

Pressure at top = 0

bottom = 7 × 30 = 210 psf

Load per linear foot

$\frac{210+0}{2} \times 7 = 735\#$

Assume this load is carried by a 4' band of reinforcing

$\frac{735}{4} = 184 \text{ lbs/ft} = .184^K$

$M = \frac{wl^2}{8} = \frac{.184 \times 23^2}{8} = 12.2 \text{ F-K}$

$A_s = \frac{M}{f_s j d} = \frac{12.2 \times 1000 \times 12}{20{,}000 \times .87 \times 12} = .7\square''/ft$

2.8□" in 4' band

use 9 #5 bars $A_s = 2.79\square''$

Section E

Reaction from end of section 'C'

$735 \times \frac{23}{2} = 8,450^{\#}$

distributed over a 3' band width $\frac{8450^{\#}}{3} = 2,850^{\#}$
per foot of width

Earth force per foot of width

maximum pressure

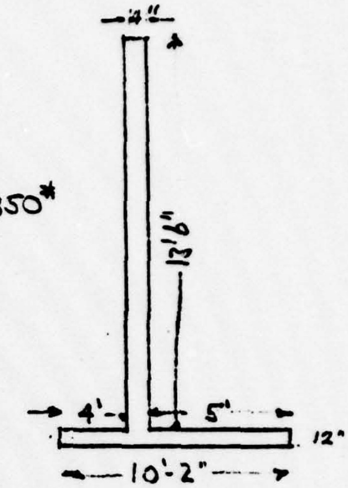
$13.5 \times 30 = 405\,psf$

total load

$\frac{405+0}{2} \times 13.5 = 2,730^{\#}$

point of action

$\frac{13.5}{3} = 4.4' = 4'$-5" up from base - add 1'0" for base = 5'5"



Stability

Resisting moments

Footing -	10.13 × 1 × 150 =	1,520# × 5.08	= 7,720'#
Stem -	13.5 × 1.13 × 150 =	2,290# × 4.58	= 10,500'#
Earth -	13.5 × 5 × 100 =	6,750# × 7.66	= 51,700'#
		10,560	69,920'#

Overturning moments

Earth	2,730# × 5.4 =	14,700'#
Reaction from 'C'	2,850# × 12.0 =	34,200'#
		48,900'#

Safety factor = $\frac{69,920}{48,900} = 1.43$

Extend heel to 6' long

Added resisting moments

Earth	13.5 × 1 × 150 = 2,020# × 10.66 =	21,400'#
Concrete	1 × 1 × 150 = 150 × 10.66 =	1,600'#
		23,000'#

added wt. = 2,020
150
2,170

Total resisting moment 69,920
23,000
92,920'#

Safety factor $\frac{92,920}{48,900} = 1.9$ - O.K.

Location of resultant

$$\begin{array}{r} 92,920 \\ 48,900 \\ \hline 44,020 \end{array}$$

Total weight

$$\begin{array}{r} 10,560 \\ 2,170 \\ \hline 12,730 \end{array}$$

$\frac{44,020}{12,730} = 3.46'$ from tip of toe

Pressure on soil

$\frac{11.16}{2} - 3.46 = 2.12 > \frac{11.16}{6} = 1.86$ hence force lies outside of middle third

$$p = \frac{P}{A} \pm \frac{Mc}{I} = \frac{P}{bh} \pm \frac{6Pe}{bh^2} = \frac{12,730}{1 \times 11.16} \pm \frac{6 \cdot 12,730 \cdot 2.12}{1 \times 11.16 \cdot 11.16}$$

$p = 1,140 \pm 1,300$

$p = 2,440$ psf or $\underline{-160 \text{ psf}}$ at end of heel

Reinforcing at base of stem

$$\begin{aligned} M - 2730 \times 4.4' &= 12,000'^{\#} \\ 2,850 \times 11.0 &= \underline{31\,400'^{\#}} \\ & \ 43,400'^{\#} = 43.4 \text{ F-K.} \end{aligned}$$

$$A_s = \frac{M}{f_s jd} = \frac{43\,400 \times 12}{20,000 \times .87 \times 12}$$

$A_s = 2.5$ sq.in/ft — too much

Change the layout of the retaining wall so that the supporting band E is 2' wide and put a 3' buttress behind it. Effective depth is 48" Total load from band "c" is now spread over 2' — hence reaction is

$\frac{8450}{2} = 4225^{\#}$ moment from reaction is $4225 \times 11 = 46,500'^{\#}$

Total moment is $46,500 + 12000 = 58,500'^{\#}$

$$A_s = \frac{M}{F_s jd} = \frac{58,500 \cdot 12}{20,000 \cdot .87 \times 48} = 0.84^{\square\prime\prime}/ft \text{ or } 1.68^{\square\prime\prime} \text{ total}$$

for buttress — use $\underline{6^{\#}5 \text{ bars}}$ $A_s = 1.86^{\square\prime\prime}$

Location of cut off for 2 bars

A_s for 4 bars is $1.24^{\square}$" or $0.62^{\square}$" for 1 ft width

$0.62 = \frac{M}{20{,}000 \times .87 \times 48}$, $M = 0.62 \times 20{,}000 \times .87 \times 48$

$M = 516{,}000$"# $= 43{,}000$'#

Bond

$u = \frac{V}{\Sigma_o j d} = \frac{6955}{5.9 \times .87 \times 48}$

$u = 283$ psi – o.k.

Try 4' above top of footing
Moment due to reaction from 'C'
$4225 \times 7 = 29{,}600$'#
Moment due to earth
$\frac{9.5 \times 30}{2} \times 9.5 \times 3.2 = 4{,}330$'#

Total moment is $4{,}330 + 29{,}600 = 33{,}930$'# $< 43{,}000$'#

Try 3' above top of footing
Moment due to reaction from 'C'
$4{,}225 \times 8 = 33{,}800$'#
Moment due to earth
$\frac{10.5 \times 30}{2} \times 10.5 \times 3.5 = 5800$'#
Total moment is $5{,}800 + 33{,}800 = 39{,}600$'# $< 43{,}000$'#

Try 2.5' above top of footing.
Moment due to reaction from 'C'
$4225 \times 8.5 = 35{,}900$'#
Moment due to earth
$\frac{11 \times 30}{2} \times 11 \times 3.67 = 6{,}650$'#
Total moment $= 35{,}900 + 6{,}650 = 42{,}550$'#

Add 24 bar diameters – $24 \times \frac{5}{8} = 15$" 15" + 30" = 45"

Hence stop 2 of the bars 3'-9" from top of footing.

Shear at base of section
horizontal thrust due to earth
$V = \frac{13.5 \times 30}{2} \times 13.5 = 2730$#

horizontal thrust from C
4225#

Total
2730
4225
6,955#

Comm. No. N-369 FRAIOLI-BLUM-YESSELMAN, CONSULTING ENGINEERS Sheet No. 9/13

fs 20,000 Archt. Date 9/25/56

fc 1250 Project Subject Des. By EDR

Check shear

$$v = \frac{V}{bjd} = \frac{6955}{12 \times .87 \times 48} = 13.9 \text{ psi} - \text{o.k.}$$

Toe of footing

Loads

Soil pressure change

Concrete

$2440-(-16)=2600$

$1'\text{-}0'' \times 150 = 150$ psf

$\frac{2600}{11.16} = 233^{\#}/ft$

at toe $2440-150=1990$ psf

under face of stem

$1990-(4\times233)=1058$ psf

Shear

$$V = 4 \times \frac{1990+1058}{2} = 6{,}100^{\#}$$

$$v = \frac{V}{bjd} = \frac{6{,}100}{12 \times .87 \times 10} = 58.5 \text{ psi} - \text{o.k.}$$

Bond

$$u = \frac{V}{\Sigma_o jd} = \frac{6100}{5.9 \times .87 \times 10}$$

$u = 119$ psi - o.k.

Moment about face of stem

$$M = 1058 \times 4 \times 2 = 8{,}460'^{\#}$$

$$= \frac{932}{2} \times 4 \times 2.67 = 4{,}970'^{\#}$$

$$13{,}430'^{\#}$$

$$A_s = \frac{M}{f_s jd} = \frac{13{,}430 \times 12}{20{,}000 \times .87 \times 10} = .926^{\square''}/ft = 1.85^{\square''} \text{ under buttress}$$

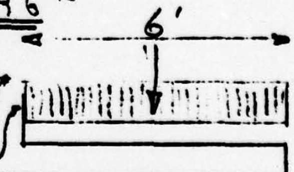
use 6 #5 bars $A_s = 1.86^{\square''}$

Heel of footing

$6 \times 13.5 \times 100 = 8100^{\#}$

$1 \times 6 \times 150 = 900$

$9{,}000^{\#}$



Moment about back of stem

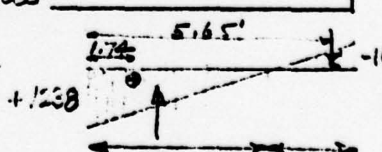
$9000 \times 3 = 27{,}000'^{\#}$

$\frac{160}{2} \times .68 \times 5.65 = 307'^{\#}$

$27{,}307'^{\#}$ $+27{,}307'^{\#}$

$\frac{1238}{2} \times 5.32 \times 1.74 = -5720'^{\#}$ $-5{,}720'^{\#}$

$+21{,}587'^{\#}$



$$A_s = \frac{M}{f_s jd} = \frac{21{,}587 \times 12}{20{,}000 \times .87 \times 10} = 1.49^{\square''}/ft = 2.98^{\square''} \text{ in the buttress band}$$

Use 5 #7 $A_s = 3.00^{\square''}$

check shear

$$V = 9000 + \left(\frac{160}{2} \times .68\right) - \left(\frac{1238}{2} \times 5.32\right) =$$
$$= 9000 + 58.5 - 3290 = 5768^{\#}$$

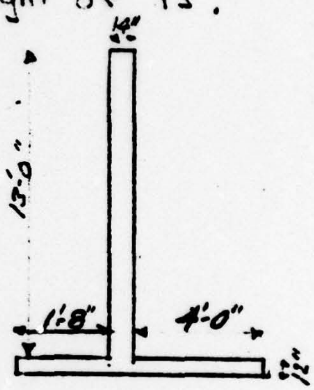
$$v = \frac{V}{bjd} = \frac{5768}{12 \times .87 \times 10} = 55.2 \text{ psi - o.k.}$$

bond

$$u = \frac{V}{\Sigma_o jd} = \frac{5768}{6.6 \times .87 \times 10}$$

$u = 100$ psi - o.k.

Design of section: A & D - both will have the same steel and will be designed for a height of 13'.



stability
Resisting moment
Footing - $6.83 \times 1 \times 150 = 1025^{\#} \times 3.41 = 3049'^{\#}$
Stem - $13 \times 1.16 \times 150 = 2260^{\#} \times 2.3 = 5200'^{\#}$
Earth - $4 \times 13 \times 100 = 5200^{\#} \times 4.83 = 25{,}100'^{\#}$
$W = 8{,}485^{\#}$ $M = 33{,}349^{\#}$

Overturning moment

$$\frac{13 \times 30}{2} \times 13 \times 4.33 = 11{,}000'^{\#}$$

Location of resultant

33,349
11,000
22,349'#

$$\frac{22{,}349'^{\#}}{8{,}485^{\#}} = 2.64' \text{ from toe}$$

Factor of safety

$$\frac{33349}{11{,}000} = 3.03 \text{ - o.k.}$$

Soil pressure
Eccentricity of resultant

$$\frac{6.83}{2} - 2.64 = 0.77 < \frac{6.83}{6}$$ hence resultant lies in middle third.

$$P = \frac{P}{A} \pm \frac{Mc}{I} = \frac{P}{bh} \pm \frac{6Pe}{bh^2} = \frac{8485}{1 \times 6.83} \pm \frac{6 \times 8485 \times .77}{1 \times 6.83 \times 6.83} = 1240 \pm 840$$

Toe pressure = 1240 + 840 = 2080 psf
heel pressure = 1240 - 840 = 400 psf

Shear at base of stem

$$V = \frac{13 \times 30}{2} \times 13 = 2540^{\#}$$

$$v = \frac{V}{bjd} = \frac{2540}{12 \times .87 \times 12} = 20.3 \text{ psi} - \text{O.K.}$$

Moment at base of stem

$$M = \frac{2540 \times 13}{3} = 11,000'^{\#}$$

$$A_s = \frac{M}{f_s jd} = \frac{11,000 \times 12}{20,000 \times .87 \times 12} = .632 \,\square''/ft - \text{use } \#5 @ 6'' \text{ o/c}, A_s = .62 \,\square''/ft$$

Location of cut-off point for half of bars

$$A_s = .31 \,\square''/ft$$

$$.31 = \frac{M}{20,000 \times .87 \times 12} = 64,900''^{\#}$$

$$M = 5,400'^{\#}$$

Try 2' up from top of footing

$$M = \frac{11 \times 30}{2} \times 11 \times 3.67 = 6,650'^{\#} - \text{too big}$$

Try $2\frac{1}{2}'$ up from top of footing

$$M = \frac{10.5 \times 30}{2} \times 10.5 \times 3.5 = 5,790'^{\#} \text{ too big}$$

Try 3' up from top of footing

$$M = \frac{10 \times 30}{2} \times 10 \times 3.3 = 4,950'^{\#} - \text{O.K}$$

add 24 bar diameters $24 \times \frac{5}{8} = 15'' = 1'\text{-}3''$

$ + 3$

Cut off $\frac{1}{2}$ of steel 4'-3" from top of footing $\quad 4'3''$

bond on steel

$$u = \frac{V}{\Sigma o jd} = \frac{2540}{3.9 \times .87 \times 12} = 62.3 \text{ psi} - \text{O.K}$$

Toe

Loads – Concrete $1 \times 150 = 150$ psf
at toe $2080 - 150 = 1930$ psf
face of stem $1930 - (1.67 \times 246) = 1520$ psf

Slope of pressure diagram

$$\begin{array}{r} 2080 \\ \underline{400} \\ 1680 \end{array}$$

$$\frac{1680}{6.83} = 246 \text{ psf/ft}$$

shear

$$V = 1.67 \times \left(\frac{1930 + 1520}{2}\right) = 2880^{\#}$$

$$v = \frac{V}{bjd} = \frac{2880}{12 \times .87 \times 10} = 27.5 \text{ psi – o.k.}$$

moment about face of stem

$$M_1 = 1520 \times 1.67 \times .83 = 2100'^{\#}$$
$$M_2 = 410 \times \frac{1.67}{2} \times 1 = \underline{340'^{\#}}$$
$$2440'^{\#}$$

$$A_s = \frac{M}{f_s jd} = \frac{2440 \times 12}{20{,}000 \times .87 \times 10} = .1685 \text{ in}^2/\text{ft} - \text{use } \#3 @ 8'' \text{ o/c}, A_s = .17 \text{ in}^2/\text{ft}$$

bond

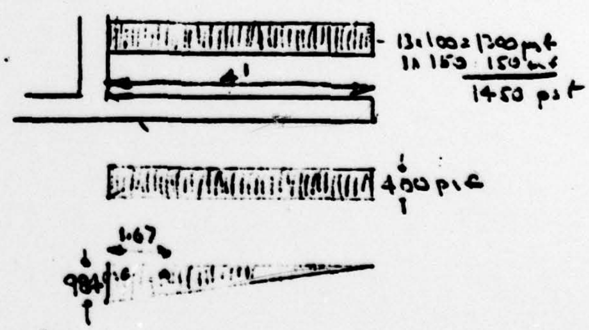
$$u = \frac{V}{\Sigma_o jd} = \frac{2880}{1.8 \times .87 \times 10} = 184 \text{ psi – o.k.}$$

Heel

shear

$\downarrow = 1450 \times 4 = 5800^{\#}$
$\uparrow = 400 \times 4 = 1600^{\#}$ 1600
$\uparrow = 984 \times 4 \times \frac{1}{2} = 1968^{\#}$ $\underline{1968}$
5800 $3568^{\#}$

$$V = \begin{array}{r} 5800 \\ \underline{-3568} \\ 2232^{\#} \end{array}$$



$$v = \frac{V}{bjd} = \frac{2232}{12 \times .87 \times 10} = 21.5 \text{ p.s.i. – o.k.}$$

Moment about face of stem

$M_+ = 1450 \times 4 \times 2 = 11{,}600'^{\#}$
$M_- = 400 \times 4 \times 2 = 3{,}200'^{\#}$ 3200
$M_- = 984 \times \frac{4}{2} \times 1.67 = 3{,}290'^{\#}$ $\underline{3290}$
$6490'^{\#}$

$$M = 11{,}600 - 6490 = 5110'^{\#}$$

$$A_s = \frac{M}{f_s j d} = \frac{5110 \times 12}{20{,}000 \times .87 \times 10} = .352^{\square''}/ft$$ use #4 @ 6" o/c bar @ 4" o/c

Bond

$$u = \frac{V}{\Sigma_0 j d} = \frac{2232}{3.1 \times .87 \times 10} = 82.7\text{ psi} - \text{o.k}$$

Design of spillway slab & supporting beam

span of slab = 8'
thickness = $11\frac{1}{2}$"
depth of water = 7'
Use 1 foot thick strip

Water load = 7×62.4 = 436.8
D.L. equal $\frac{11.5}{12} \times 150$ = 143.7
580.5 lb/ft

$$M = \frac{w l^2}{8} = \frac{580.5 \times 8^2}{8} = 4644'$$

$$A_s = \frac{M}{f_s j d} = \frac{4644 \times 12}{20{,}000 \times .87 \times 10} = .32$$

use #3 @ 4" o/c, $A_s = .33^{\square''}/'$

Span of beam = 17'-6"
load per foot coming in from slab
Water - 5'×7'×62.4 = 2190#
Slab - $4 \times \frac{11.5}{12} \times 150$ = 575#
beam - 2.25×1.5×150 = 506#
3271#/ft of beam

$$M = \frac{1}{8} w l^2 = \frac{1}{8} \times 3271 \times \overline{17.5}^2 = 125{,}000'^{k}$$

$$A_s = \frac{M}{f_s j d} = \frac{125{,}000 \times 12}{20{,}000 \times .87 \times 24} = 3.6^{\square''}$$ use 3 #10 - $A_s = 3.8^{\square''}$

Check for shear

$$V = 3271 \times \frac{17.5}{2} = 28{,}600^{\#}$$

$$v = \frac{V}{b j d} = \frac{28.600}{19 \times .87 \times 24} = 72.2\text{ psi} < 75\text{ psi} - \text{o.k.}$$

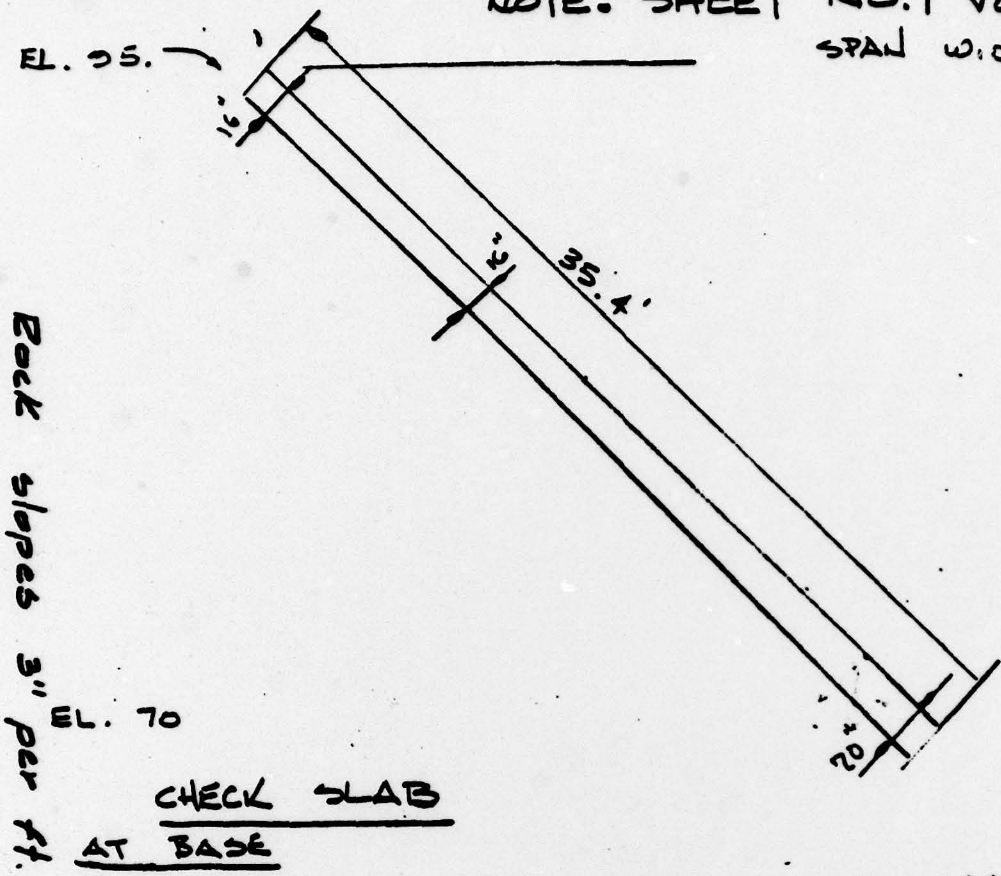
Comm. No. N247 FRAIOLI-BLUM-YESSELMAN, CONSULTING ENGINEERS Sheet No. 2

fs. 369 Archt. CAUSEY & WEEKS Date 27AUG

fs. Project FALL. CR. DAM Subject SLAB Des. By RCO

NOTE: SHEET NO. 1 VOIDED

SPAN WIDTH = 15.92'



CHECK SLAB
AT BASE

P = .0624(25) = 1.56
SLAB = 1.67(15)/1.414 = .18
w LOAD = 1.74

M = 1.74(15.92)²/8 = 55.2 ^1-K/1
As = 55.2 / 1.44(17) = 2.28
As FURNISHED — 2.37

AT ELEV. 74.0
P = .0624(21) = 1.31
SLAB = .18
w = 1.49

M = 1.49(15.92)²/8 = 47.5
As = 47.5 / 1.44(16) = 2.07
As furn = 2.11

AT ELEV. 77.0
P = .0624(18) = 1.12
SLAB = 1.58(.15)/1.414 = .17
w = 1.29

M = 1.29(15.92)²/8 = 41.5
As = 41.5/1.44(15.5) = 1.86
As furn = 1.90

AT ELEV. 83.0
P = .0624(12) = .75
SLAB = .17
w = .92

M = .92(15.92)²/8 = 29.5
As = 29.5/1.44(14) = 1.47
As furn = 1.72

AT ELEV. 86.5

$p = 8.5(.0624) = .53$

$SLAB = \frac{1.33(.15)}{1.414} = .14$

$w = .67$

$M = .67(15.92)^2/8 = 21.5$

$A_s = 21.5/1.44(13) = 1.15$

A_s furn 1.46

AT ELEV. 88.0

$p = 7.0(.0624) = .44$

$SLAB = .14$

$w = .58$

$M = 18.6$

$A_s = 1.0$ – USE #7@7"

AT ELEV 91.5

$p = 3.5(.0624) = .22$

$SLAB = .14$

$w = .36$

$M = 11.3$

$A_s = .71$ USE – #6@7"

SHEAR ON BUTTRESS LIP SUPPORTING SLAB

$R = 14^{k/1}$

$A = 20(12) = 144$

$v = 14000/144 = 97^{psi}/1$ – LIP HAS #6 bars – O.K.

BEARING ON LIP

$R = 14^{k/1}$

$A = 9(12) = 108$

$f_c = 14000/108 = 130$ PSI – OK

Comm. No. IV C41

fs. 369

fs. ________

FRAIOLI-BLUM-YESSELMAN, CONSULTING ENGINEERS

Archt. CAUSEY & WEEKS

Project FALL. CR. DAM

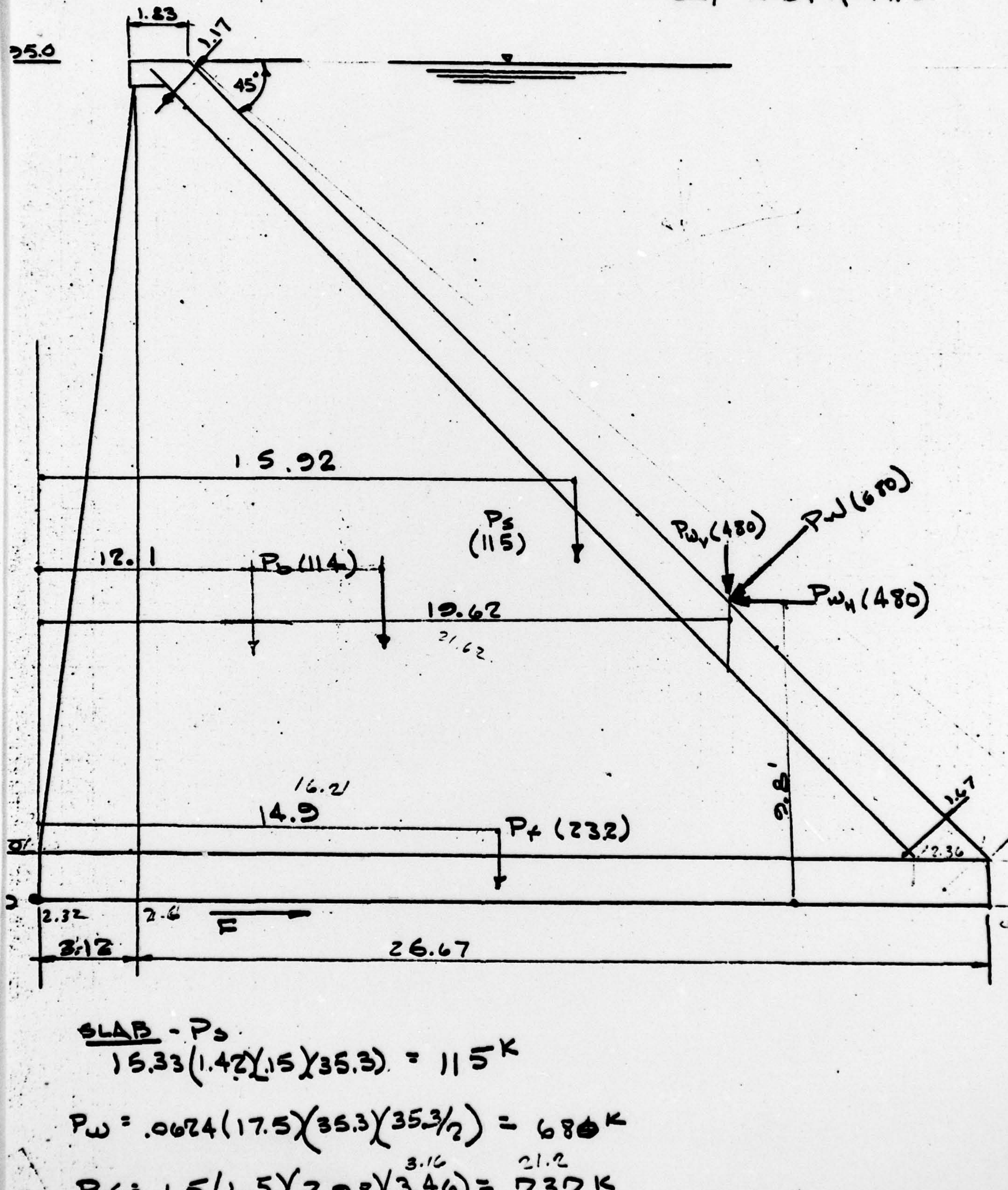
Subject STABILITY

Sheet No. 4

Date 28 AUG 56

Des. By RCO

BAY WIDTH = 17.5'



SLAB - Ps

$15.33(1.42)(.15)(35.3) = 115^K$

$P_W = .0624(17.5)(35.3)(35.3/2) = 680^K$

$P_f = 1.5(1 \times 5)(29.8)(3.46) = 23.2^K$ (3.16) (21.2)

Pb

$\frac{250}{2}(.15)(25.67)(2.13) = 103^K$

$25/2\,(.15)(1.76)(3.12) = 11^K$

$P_b = 114^K$

ΣMo

(Pb) 12.1(114) = 1380   480(9.8) = 4700
(Pt) 14.9(23.2) = 345
(Ps) 15.92(115) = 1830
(Pwr) 19.62(480) = 9420
12,975 1-k

$W/R = 8,275/832 = 9.95$ 11.3

F = ~~832~~ 732 (.65) = 540 ~~475.8~~

SLIDING S.F. = 1.13

O.T. S.F = 12,975/4700 = 2.7

SOIL PRESSURE

$f = \frac{832}{3.46} = 240$

$P_{act}\ \frac{2f}{l} = \frac{480}{29.75} = 16^{k.s.f.} = 8$ TON SQ FT

APPENDIX V

CONCRETE TEST REPORTS

FRC HLING & ROBERTSON, IC.
INSPECTION ENGINEERS • CHEMISTS • BACTERIOLOGISTS

SINCE F&R 1881

BRANCH LABORATORIES
NORFOLK, CHARLOTTE, RALEIGH
WASHINGTON, BALTIMORE

MAIN OFFICE & LABORATORIES
814 WEST CARY STREET
RICHMOND, VIRGINIA

No. E-4784-10

November 20, 1957

CONCRETE TEST REPORT

Project Addition to Filtration Works, Contract C-1, Chesterfield County, Virginia

Location Sampled Apron slab, Buttress Nos. 2 and 3 Dam

Designed Compressive Strength 3000 lbs. per sq. in. at 28 days. Date moulded 10/22/57

MATERIALS USED — PER CUBIC YARD OF CONCRETE

Cement 5.70 Sacks. Brand Lone Star Mfd. at Norfolk, Va.

Fine Aggregate 1166 Pounds. Source Size 100 to #4

Coarse Aggregate 1950 Pounds. Source Size #4 to 1"

Water 32.8 Gallons, including moisture in aggregates. Gals. water per sack 5.8

Admixture Pozzolith 3% Solution Amount 5.7 Water-Cement Ratio Slump 2-1/2"

Made for English Construction Company, Altavista, Virginia

Received 10/27/57 Condition:

Marked D17, 18, 19, 20

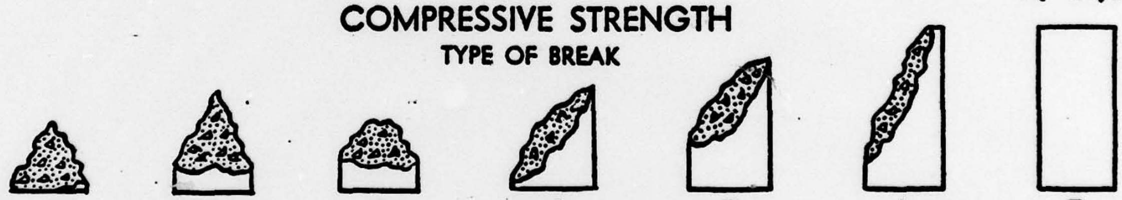
Curing: Field: to F° Period:

Damp Sand, to F° Period:

Moist Room, 73 F° Period: 2 days 23 days

COMPRESSIVE STRENGTH

TYPE OF BREAK



1 2 3 4 5 6 7

No.	Size Inches	Breaking Load Pounds	Pounds per Sq. In.	Weight Lbs. Per Cu. Ft.	Age at Test	Type of Break	Per Cent Aggregate Broken through Line of Fracture
17	6 x 12	60,500	2140	148.2	7 days	2	1
18	6 x 12	116,000	4103	147.7	28 days	1	6
19	6 x 12	114,000	4032	148.2	28 days	1	5
20	6 x 12	112,000	3961	147.7	28 days	2	5

Conclusions:

1 cc English Construction Co., Altavista, Va.
1 cc English Construction Co., Rt. 10, Box 144A, Richmond, Va.
1 cc R. Kenneth Weeks, Engineers, 6165 E. Sewell Point Rd, Norfolk, Va.
1 cc Robert Painter, Engineer, Engineers Office, Chesterfield Courthouse, Chesterfield Co. Va.
1 cc Edgar L. White, Res. Eng., 24 E. Belt Boulevard, Richmond, Va.

ENGINEER'S OFFICE NOV 20 1957 CHESTERFIELD COUNTY, VA.

FROEHLING & ROBERTSON, INC.

Form CT-57

FROEHLING & ROBERTSON, INC.
INSPECTION ENGINEERS • CHEMISTS • BACTERIOLOGISTS
SINCE F&R 1881

BRANCH LABORATORIES
NORFOLK, CHARLOTTE, RALEIGH
WASHINGTON, BALTIMORE

MAIN OFFICE & LABORATORIES
814 WEST CARY STREET
RICHMOND, VIRGINIA

No. E-4784-11 November 25, 1957

CONCRETE TEST REPORT

Project: Addition to Filtration Works, Contract C-1, Chesterfield County, Virginia

Location Sampled: Settling basins, Wall # 4, Also Dam-Beams Buttress Nos. 8 & 9 - 9 & 10

Designed Compressive Strength 3000 lbs. per sq. in. at 28 days. Date moulded 10/26/57

MATERIALS USED — PER CUBIC YARD OF CONCRETE

Cement 5.70 Sacks. Brand Lone Star Mfd. at Norfolk, Va.

Fine Aggregate 1166 Pounds. Source Size 100 to #4

Coarse Aggregate 1950 Pounds. Source Size #4 to 1"

Water 32.8 Gallons, including moisture in aggregates. Gals. water per sack 5.8

Admixture Pozzollith 3% Solution Amount 5.7 quarts Water-Cement Ratio Slump 2-1/2"

Made for English Construction Company, Altavista, Virginia

Received 10/31/57 Condition:

Marked SB Nos. 17,18,19,20

Curing: Field: to F° Period: | Damp Sand, to F° Period: | Moist Room, 73 F° Period: 2 days 23 days

COMPRESSIVE STRENGTH

TYPE OF BREAK

1 2 3 4 5 6 7

No.	Size Inches	Breaking Load Pounds	Pounds per Sq. In.	Weight Lbs. Per Cu. Ft.	Age at Test	Type of Break	Per Cent Aggregate Broken through Line of Fracture
17	6 x 12	61,000	2157	147.1	7 days	2	3
18	6 x 12	103,000	3643	147.1	28 days	1	5
19	6 x 12	104,500	3696	146.8	28 days	2	7
20	6 x 12	106,000	3749	146.8	28 days	2	5

Conclusions:

1 cc English Construction Co. Inc., Altavista, Va.
1 cc English Construction Co. Inc., Rt. 10, Box 144A, Richmond, Va.
1 cc R. Kenneth Weeks, Engineers, 6165 E. Sewells Point Rd, Norfolk, Va.
1 cc Robert Painter, Engineer, Engineers Office, Chesterfield Courthouse, Chesterfield County, V
1 cc Edgar L. White, Res. Eng., 24 E. Belt Boulevard, Richmond, Va.

ENGINEER'S OFFICE NOV 23 1957 CHESTERFIELD VA.

FROEHLING & ROBERTSON, INC.

Form CT-57

FROEHLING & ROBERTSON, INC.

INSPECTION ENGINEERS • CHEMISTS • BACTERIOLOGISTS

SINCE F&R 1881

BRANCH LABORATORIES
NORFOLK, CHARLOTTE, RALEIGH
WASHINGTON, BALTIMORE

MAIN OFFICE & LABORATORIES
814 WEST CARY STREET
RICHMOND, VIRGINIA

No. E-4734-10

November 7, 1957

CONCRETE TEST REPORT

Project Addition to Filtration Works, Contract C-1, Chesterfield County, Virginia

Location Sampled Dam-Buttress Nos. 8-9-10 to Elev. 88.0'

Designed Compressive Strength 3000 lbs. per sq. in. at 28 days. Date moulded 10/10/57

MATERIALS USED — PER CUBIC YARD OF CONCRETE

Cement 5.70 Sacks. Brand Lone Star Mfd. at Norfolk, Va.

Fine Aggregate 1166 Pounds. Source Size 100 to #4

Coarse Aggregate 1950 Pounds. Source Size #4 to 1"

Water 32.8 Gallons, including moisture in aggregates. Gals. water per sack 5.8

Admixture pozzolith 3% solution Amount 5.7 qts. Water-Cement Ratio Slump 3"

Made for English Construction Company, Altavista, Virginia

Received 10/15/57 Condition:

Marked D-13-14-15-16

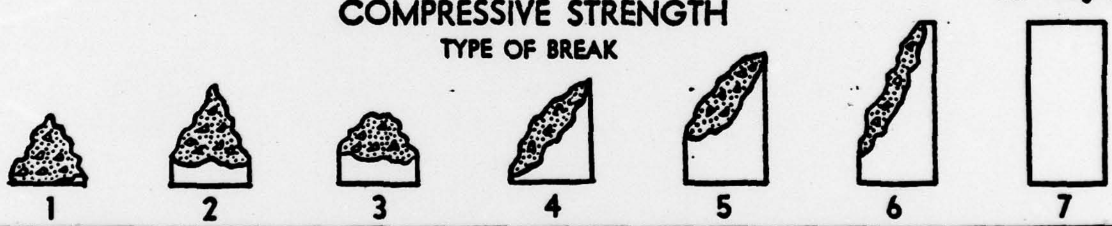
Curing: Field: to F° Period:

Damp Sand, to F° Period:

Moist Room, 73 F° Period: 2 days 23 days

COMPRESSIVE STRENGTH

TYPE OF BREAK



No.	Size Inches	Breaking Load Pounds	Pounds per Sq. In.	Weight Lbs. Per Cu. Ft.	Age at Test	Type of Break	Per Cent Aggregate Broken through Line of Fracture
D-13	6 x 12	71,000	2511	147.7	7 days	1	2
D-14	6 x 12	127,500	4510	148.3	28 days	1	8
D-15	6 x 12	124,500	4404	147.7	28 days	1	6
D-16	6 x 12	120,000	4244	147.7	28 days	2	8

Conclusions:

1 cc English Construction Co., Altavista, Va.
1 cc English Construction Co., Rt. 10, Box. 144A, Richmond, Va.
1 cc R. Kenneth Weeks, Engineers, 6165 E. Sevells Point Rd., Norfolk, Va.
1 cc Robert Painter, Engineer, Engineers Office, Chesterfield Courthouse, Chesterfield County, Va.
1 cc Edgar L. White, Res. Eng., 24 East Belt Boulevard, Richmond, Va.

FROEHLING & ROBERTSON, INC.

Form CT-57

FROEHLING & ROBERTSON, INC.

INSPECTION ENGINEERS • CHEMISTS • BACTERIOLOGISTS

SINCE F&R 1881

BRANCH LABORATORIES
NORFOLK, CHARLOTTE, RALEIGH
WASHINGTON, BALTIMORE

MAIN OFFICE & LABORATORIES
814 WEST CARY STREET
RICHMOND, VIRGINIA

No. E-4784-9

September 27, 1957

CONCRETE TEST REPORT

Project: Addition to Filtration Works, Contract C-1

Location Sampled: Dam Buttress Pour - Buttress No. 6

Designed Compressive Strength 3000 lbs. per sq. in. at 28 days. Date moulded 8/30/57

MATERIALS USED — PER CUBIC YARD OF CONCRETE

Cement 5.70 Sacks. Brand Lone Star Mfd. at Norfolk, Va.

Fine Aggregate 1166 Pounds. Source Size 100 to #4

Coarse Aggregate 1950 Pounds. Source Size #4 to 1"

Water 32.8 Gallons, including moisture in aggregates. Gals. water per sack 5.8

Admixture Pozzolith 3% solution Amount 5.70 quarts Water-Cement Ratio Slump 3"

Made for English Construction Company, Inc., Altavista, Virginia

Received 9/3/57 Condition:

Marked D-9, 10, 11 and 12.

Curing: Field: to F° Period: | Damp Sand, to F° Period: | Moist Room, 73 F° Period: 3 days 24 days

COMPRESSIVE STRENGTH

TYPE OF BREAK

1 2 3 4 5 6 7

No.	Size Inches	Breaking Load Pounds	Pounds per Sq. In.	Weight Lbs. Per Cu. Ft.	Age at Test	Type of Break	Per Cent Aggregate Broken through Line of Fracture
D-9	6 x 12	101,000	3572	147.7	7 days	1	6
D-10	6 x 12	140,100	4952	146.2	28 days	2	10
D-11	6 x 12	152,000	5376	146.7	28 days	1	14
D-12	6 x 12	137,000	4845	147.1	28 days	1	10

Conclusions:

ENGINEER'S OFFICE
SEP 30 1957
CHESTERFIELD COUNTY, VA.

1 cc English Construction Company, Altavista, Va.
1 cc English Construction Company, Rt. 10, Box 144A, Richmond, Va.
1 cc R. Kenneth Weeks, Engineers, 6165 E. Sewells Point Rd, Norfolk, Va.
1 cc Robert Painter, Engineer, Engineers Office, Chesterfield Court House, Chesterfield Co, Va.
1 cc Edgar L. White, Res. Eng., Rt. 7, Box 458, Richmond, Va.

FROEHLING & ROBERTSON, INC.

Form CT-57

APPENDIX VI

REFERENCES

APPENDIX VI

1. HEC-1DB Flood Hydrograph Package, (Hydrologic Engineering Center, U. S. Army Corps of Engineers, July 1978).

2. "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian," Hydrometeorological Report No. 33, (U. S. Weather Bureau, April 1956).

3. "Rainfall Frequency Atlas of the United States," Technical Paper No. 40, (U. S. Weather Bureau, May 1961).

4. Recommended Guidelines for Safety Inspection of Dams, Office of the Chief of Engineers, Department of the Army, Washington, D. C.

5. INSPECTION REPORT SWIFT CREEK DAM AND FALLING CREEK DAM, CHESTERFIELD CO., VA., J. K. Timmons & Associates, Inc., March, 1978.